PROCEEDINGS

WORKSHOP ON HYDRAULICS OF RIVER ICE

June 20 and 21, 1984
Fredericton, New Brunswick
Canada

COMPTE RENDU

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU

les 20 et 21 Juin 1984
Fredericton, Noveau-Brunswick
Canada

Compiled by / Composé par
K.S. Davar and B.C. Burrell

SPONSORS

N.R.C.C. SUBCOMMITTEE ON HYDRAULICS OF ICE COVERED RIVERS

NEW BRUNSWICK SUBCOMMITTEE ON RIVER ICE

UNIVERSITY OF NEW BRUNSWICK

PARRAINAGE

LE SOUS-COMITÉ DU C.N.R. SUR L'HYDRAULIQUE DES COURS D'EAU RECOUVERTS DE GLACE

LE SOUS-COMITÉ DU NOVEAU BRUNSWICK SUR LA GLACE DES COURS D'EAU

L'UNIVERSITÉ DU NOVEAU-BRUNSWICK

ORDERING PROCEDURE

A limited number of copies has been printed and are available for purchase at $30.00 per copy (prepaid in Canadian funds) from:

Third Workshop on Hydraulics of River Ice
c/o Dr. K.S. Davar, Professor
Department of Civil Engineering
University of New Brunswick
Fredericton Campus
P.O. Box 4400
Fredericton, N.B.
CANADA
E3B 5A3

POUR COMMANDER

Les Actes des travaux de l'Atelier sont disponsibles au prix au prix de 30 $ CDN l'exemplaire et le nombre est limité; pour s'en procurer, envoyer un ordre de paiement en dollars canadiens à:

Troisième Atelier sur l'hydraulique de la glace des cours d'eau
a/s M. K.S. Davar, Professeur
Departement de génie Civil
University of New Brunswick
Campus de Fredericton
C.P. 4400
Fredericton, N.-B.
CANADA
E3B 5A3

The papers, discussions, and poster display abstracts contained herein are published in the form presented by the authors. No attempt has been made to alter the material except where obvious errors or discrepancies were detected, or where rewording was needed for the convenience of the reader. Any statements or views here presented are totally those of the authors, and are not condoned or rejected by any of the sponsoring agencies. Mention of trade names, commerical products or firm names does not constitute a recommendation for use.

Les documents, les compte rendus de discussions et les résumés de placards publicitaires ci-inclus sont publiés de la façon dont ils ont été présentés par leurs auteurs. Aucune tentative n'a été effectuée pour modifier la documentation, sauf lorsque l'on a détecté des erreurs ou des divergences évidentes ou lorsqu'il était nécessaire de remanier le texte pour en faciliter la lecture. Toutes les déclarations ou les opinions présentées sont totalement celles des auteurs et elles ne sont pas rejetées par les organismes de parrainage. La mention de marques déposées, de produits commerciaux ou de noms de firmes ne constitue pas une recommandation d'emploi.

ACKNOWLEDGEMENTS

The Workshop on Hydraulics of River Ice was sponsored by the N.R.C.C. Subcommittee on Hydraulics of Ice-Covered Rivers; in conjunction with the New Brunswick Subcommittee on River Ice and the University of New Brunswick.

Financial assistance was obtained from the New Brunswick Electric Power Commission, the New Brunswick Department of the Environment, and the University of New Brunswick.

The New Brunswick Department of the Environment and the Inland Waters Directorate (Atlantic Region) of Environment Canada purchased copies of these proceedings for placement in a few selected government and educational libraries in Atlantic Canada. Their advance purchase helped greatly in meeting the financial obligations of the Workshop.

Thanks are expressed to all the agencies who allowed staff to become involved with the organization of the Workshop.

Member agencies of the New Brunswick Subcommittee on River Ice are thanked for providing some translation services.

The New Brunswick Department of Tourism and the Tourism and Conventions Department of the City of Fredericton provided information on tourist accommodation and attractions, which proved useful to several people attending the Workshop.

The New Brunswick Department of Transportation kindly provided signs which helped direct people to the Workshop.

During the Workshop, staff at the Registration Desk were provided by the Department of Civil Engineering of the University of New Brunswick. Gratitude is also expressed to J.E. Peters Management Limited, who provided an employee to staff the Information Desk.

Staff of the University of New Brunswick and the New Brunswick Department of the Environment are thanked for their help during the compilation and preparation of these proceedings.

GENERAL ORGANIZING COMMITTEE

Dr. K.S. Davar, Professor (Chairman)
Dept. of Civil Engineering
University of New Brunswick
Fredericton Campus
College Hill Road
P.O. Box 4400
Fredericton, N.B.
E3B 5A3

Dr. D.I. Bray, Professor (Member)
Dept. of Civil Engineering
University of New Brunswick
Fredericton Campus
College Hill Road
P.O. Box 4400
Fredericton, N.B.
E3B 5A3

Mr. J.G. Lockhart, Director (Member)
Saint John River Forecast Centre
c/o Dept. of the Environment
P.O. Box 6000
Fredericton, N.B.
E3B 5H1

Mr. B.C. Burrell, P. Eng. (Member & Secretary)
Hydrotechnical Engineer
Water Resources Branch
Dept. of the Environment
P.O. Box 6000
Fredericton, N.B.
E3B 5H1

Dr. Sayed Ismail, P. Eng. (Member)
System Planning Division
N.B. Electric Power Commission
P.O. Box 2000
505 King St.
Fredericton, N.B.
E3B 4X1

THE TECHNICAL PROGRAM SUBCOMMITTEE

Brian Burrell, P. Eng.*	N.B. Dept. of the Environment
John Dublin	A.E.S., Environment Canada
Pat Tang, P. Eng.	N.B. Dept. of the Environment
Michael Riley, P. Eng.	Geocon Inc.
Jean-Guy Deveau, P. Eng.	I.W.D., Environment Canada
Dave Randall, P. Eng.	I.W.D., Environment Canada

THE REGISTRATION AND BUDGET SUBCOMMITTEE

Dale Bray, P. Eng.*	University of New Brunswick
Pat Tang, P. Eng.	N.B. Dept. of the Environment

THE SOCIAL EVENTS SUBCOMMITTEE

Jerry Lockhart, P. Eng.*	Saint John River Forecast Centre
Nabil Elhadi, P. Eng.	N.B. Dept. of the Environment
John Dublin	A.E.S., Environment Canada

THE SHORT COURSE SUBCOMMITTEE

Sayed Ismail, P. Eng. *	New Brunswick Electric Power Comm.
Daryll DeMerchant, P. Eng.	ADI Ltd.
David Sullivan, P. Eng.	N.B. Dept. of Transportation

*Chairmen

WORKSHOP CHAIRMAN:	Dr. K.S. Davar
ASSISTANT WORKSHOP CHAIRMEN:	Mr. J.-G. Deveau Mr. B.C. Burrell
SESSION CHAIRMEN:	Dr. N. Elhadi Mr. D. Andres Dr. B. Pratte Mr. J.G. Lockhart
WRAP-UP SPEAKERS:	Dr. R. Gerard Mr. D. Calkins
STUDENT ASSISTANTS:	Mr. F. Hendrickson Mr. J. Ho Mr. I. MacGougan Mr. D. Rickemann
REGISTRATION DESK:	Mrs. G. Stephens Mrs. D. Lewis Mrs. T. Mofford
INFORMATION DESK:	Ms. M. LeBrun-Salonen

REMERCIEMENTS

L'Atelier sur l'hydraulique de la glace des cours d'eau a été organisé par le sous-comité du C.N.R.C. de l'hydraulique des cours d'eau couverts de glace, en collaboration avec le sous-comité du Nouveau-Brunswick sur la glace des cours d'eau et la University of New Brunswick.

La Commission d'énergie electrique du Nouveau-Brunswick, le ministère de l'Environnement du Nouveau-Brunswick et la University of New Brunswick ont participé financièrement a l'organisation de l'Atelier.

Le ministère de l'Environnement du Nouveau-Brunswick et Direction générale des eaux intérieures-Atlantique d'Environnement Canada ont acheté des exemplaires des Actes de l'Atelier à l'intention de certaines bibliothèques du gouvernement ou d'institutions d'enseignement dans les provinces de l'Atlantique. L'achat anticipé de ces publications a été d'un grand secours à l'Atelier.

Nos sincères remerciements s'adressent à tous les organismes qui ont permis à des employés de travailler à l'organisation de l'Atelier.

Nous remercions les organismes membres du sous-comité du Nouveau-Brunswick sur la glace des cours d'eau d'avoir bien voulu fournir des services de traduction.

Le ministère du Tourisme du Nouveau-Brunswick et le Service du tourisme et des congrès de la ville de Fredericton ont donné des renseignements sur le logement et les attractions touristiques dont plusieurs participants a l'Atelier ont pu profiter.

Le ministère des Transports du Nouveau-Brunswick a eu la gentillesse de placer des affiches pour diriger les participants a l'Atelier.

Au cours de l'Atelier, le personnel du bureau d'inscription ont été prêté par le département de génie civil de la University of New Brunswick. Nos remerciements s'adressent également à la société J.E. Peters Management Ltd., qui a fourni un employé au Bureau d'information.

Le personnel de la University of New Brunswick et du ministère de l'Environnement du Nouveau-Brunswick ont droit à nos remerciements chaleureux pour avoir contribué à la compilation et à la préparation des Actes de l'Atelier.

COMITE ORGANISATEUR

Dr. K.S. Davar, professeur (président)
Department de génie civil
University of New Brunswick
Campus de Fredericton
C.P. 4400
Frédéricton, (N.-B)
E3B 5A3

Dr. D.I. Bray, professeur (membre)
Departement de génie civil
University of New Brunswick
Campus de Fredericton
C.P. 4400
Frédéricton (N.-B)
E3B 5A3

M. J.G. Lockhart, Directeur (membre)
Centre de prévision des cures de
la rivière Saint-Jean
a/s Ministère de l'environnement
C.P. 6000
Frédéricton (N.-B)
E3B 5H1

M. B.C. Burrell, ing. (membre et secrétaire)
Ingénieur hydrotechnique
Direction des resources en eau
Ministère de l'environnement
C.P. 6000
Frédéricton (N.-B)
E3B 5H1

Dr. Sayed Ismail, ing. (membre)
Division des systèms de gestion
Commission d'énergie électrique du
Nouveau Brunswick
C.P. 2000
500, rue King
Frédéricton (N.-B)
E3B 4X1

SOUS-COMITÉ DE LA PROGRAMMATION

Brian C. Burrell, ing.*	Ministère de l'environnement
John Dublin	S.A.E., Environnement Canada
Pat Tang, ing.	Ministère de l'environnement
Michael Riley, ing.	Geocon Inc.
Jean-Guy Deveau, ing.	D.G.E.I., Environnement Canada
Dave Randall, ing.	D.G.E.I., Environnement Canada

SOUS-COMITÉ DE L'INSCRIPTION ET DU BUDGET

Dale Bray, ing.*	University of New Brunswick
Pat Tang, ing.	Ministère de l'environnement

SOUS-COMITÉ DU PROGRAMME SOCIAL

Jerry Lockhart, ing.*	Centre de prévision des crues de la rivière Saint-Jean
Nabil Elhadi, ing.	Ministère de l'environnement
John Dublin	S.A.E., Environnement Canada

SOUS-COMITÉ DU MINI COURS

Sayed Ismail, ing.*	Commission d'éngerie électrique de Nouveau-Brunswick
Daryll DeMerchant, ing.	ADI Ltd.
David Sullivan, ing.	Ministère de Transportation

*Présidents

Président de l'atelier:	Dr. K.S. Davar
Président adjoint de l'atelier:	M. J.-G. Deveau M. B.C. Burrell
Animateur de session:	Dr. N. Elhadi M. D. Andres Dr. B. Pratte M. J.G. Lockhart
Orateurs de cloture:	Dr. R. Gerard M. D. Calkins
Aides étudiants:	M. F. Hendrickson M. J. Ho M. I. MacGougan M. D. Rickemann
Inscription:	Mme. G. Stephens Mme. D. Lewis Mme. T. Mofford
Information:	Ms. M. LeBrun-Salonen

TABLE OF CONTENTS/TABLE DES MATIERES

	Page
Disclaimer	iii.
Acknowledgements	iv.
Remerciements	viii.
Table of Contents/Table des Matieres	xii.
Opening Remarks	1
Allocution d'Ouverture	2
PART ONE - TECHNICAL PAPERS PREMIERE PARTIE-EXPOSES TECHNIQUES	
Session A - Estuaries/Estuaires	
Winter ice regime in the tidal estuaries of the Northeastern Portion of the Bay of Fundy, N.B. -C. Desplanque and D.I. Bray	7
Session B - Breakup/Debacle	
Study of river ice breakup using hydrometric station records -Dr. S. Beltaos	41
Forecasting the initiation of ice breakup on the Nashwaak River, NB -P.W. Tang and K.S. Davar	65
Hydraulics of breakup -H.S. Santeford and G.R. Alger	95

Session C - Ice Jams/Embacles de Glace

Case studies concerned with ice jamming
-S. Petryk 113

Cutting trenches in the ice cover to prevent ice jams
-L. Jolicoeur, B. Michel and J. Labbé 127

Documentation and analysis of the water level profile through an ice jam, MacKenzie River, N.W.T.
-G. Rivard, T. Kemp and R. Gerard 141

Protection Contre les inondations en presence des glaces, Rivière Saint-Charles a Quebec
-J. Monfet et A.R. Tremblay 159

Ice Jam research needs
-R. Gerard 181

Session D - Flow Conveyance and Resistance/ Transfert et Resistance des Ecoulements

Iceforms of floating loose covers
-S.P. Chee and M. R. Haggag 197

Coverage coefficient for calculating ice volume generated
-R. Hausser, J.P. Saucet, and F.E. Parkinson 211

Session E - Modelling/Modellage

Computer simulation of ice cover formation in the Upper St. Lawrence River
-H.T. Shen and P.D. Yapa 227

Numerical simulation of freeze-up on the Ottauquechee River
-D.J. Calkins 247

Winter rating curves and ice volume limited water levels
-S.T. Lavender 279

Session F - General/Generalité

The rise pattern and velocity of frazil ice
-J.L. Wuebben 297

Measurements of frazil and drifting ice in the Lachine Rapids
-B. Michel, P. Desroches, J.G. Dussault, F. Fonseca, and D. Levan 317

Anchor ice effects on water levels in Lake St. Louis, St. Lawrence River at Montreal
-F.E. Parkinson 345

Improved ice control with better understanding of thermal regime of the international section of St. Lawrence River
-D. Bunte-Bisnett, S.K. Gupta and R.W. Penn 371

Interaction between surface waves and a densely packed broken ice cover
-E.P. Foltyn, and C.D. Ponce-Campos 387

PART TWO - ABSTRACTS FOR POSTER DISPLAYS
DEUXIEME PARTIE-SOMMAIRES DES AFFICHES A EXPOSER

New Brunswick's Subcommittee on River Ice
Sous-Comité du Nouveau Brunswick sur la glace des Cours d'eau
-B. Burrell 409

River ice instrumentation developments at the National Water Research Institute
Mises au point d'instruments d'analyse de la glace fluviale a l'Institut national de recherche sur les eaux
-J.S. Ford, J. Valdmanis, A.S. Watson 413

Ice Jams in New Brunswick
Embâcles de glace au Nouveau-Brunswick
-N. Elhadi, G. Richard 415

The Canadian Climate Centre Archive
Les Archives du centre climatique Canadien
-J. Dublin, S.F. Malone 419

Exploits River freeze-up and break-up observations 1984
Exploits cours d'eau de la glace et la debacle observations 1984
-D.B. Hodgins, H.L. Davis 421

A field survey form for monitoring ice conditions on lakes and rivers
Formule d'observations sur l'état de la glace de lac et de rivière
-T.D. Prowse and J.C. Anderson 425

Freeze-up and break-up processes on the Yukon River in the vicinity of Dawson City, Yukon Territory
Prise de la glace et la debacle sur la rivière du Yukon dans les environs de la ville de Dawson, Territoire du Yukon
-J.R. Janowicz 431

The NRCC Working Group in River Ice Jams
-S. Beltaos 434

Closing Remarks 437

Allocution De Cloture 438

ANNEXE A - ORGANISATEURS DE L'ATELIER 439
APPENDIX A - WORKSHOP SPONSORS

ANNEXE B - LISTE DES PARTICIPANTS 445
APPENDIX B - LIST OF PARTICIPANTS

OPENING REMARKS

Greetings and Welcome to the 3rd Workshop

This 3rd Workshop on Hydraulics of River Ice represents an ongoing effort across Canada to focus attention on a subject area of considerable hydroeconomic importance for management of rivers during the winter regime. The workshops are intended to provide an opportunity for presentation of latest developments in theory, analytical techniques, and application, also; to provide a forum for free discussion which, hopefully, will lead to useful conclusions about future directions.

For the first time, the Workshop has been preceded by a two-day Short Course on River Ice Engineering. The Short Course has been designed for personnel from river management agencies, and consultants; the aim was to review applications methodology and present proven techniques for practical problem solving. It's aims are complementary to those of the Workshop.

An important feature of the Workshop is to provide equal time allocations for presentation of papers and subsequent discussions; clearly, the discussions have been given as much importance as the papers. May I urge participants to use this opportunity freely for spontaneous, interesting and useful contributions.

This 3rd Workshop on the Hydraulics of River Ice has been jointly sponsored by the following organizations:

N.R.C. Subcommittee on Hydraulics of River Ice;

N.B. Subcommittee on River Ice;

University of New Brunswick.

After the sessions, may I invite you to discover and enjoy the several scenic locations on our campus, the interesting historical, and cultural institutions in the City of Fredericton, and travel and relax in our beautiful province.

K.S. Davar, Chairman
General Organizing Committee

ALLOCUTION D'OUVERTURE

Bonjour et bienvenue à tous au 3e Atelier

Le 3e Atelier sur l'hydraulique de la glace des cours d'eau est la manifestation d'efforts incessants d'un bout à l'autre du Canada en vue d'appeler l'attention sur un domaine d'une grande importance hydroéconomique pour la gestion des cours d'eau pendant le régime hivernal. Les ateliers ont un double but, soit de fournir une occasion de communiquer les faits nouveaux sur la théorie, les techniques d'analyse et les applications dans le domaine, et de fournir une tribune pour la discussion qui, nous le souhaitons, débouchera sur des conclusions quant aux orientations futures.

L'Atelier a été précédé, pour la première fois d'un programme abrégé de deux jours sur les applications techniques touchant la glace des cours d'eau; le programme abrégé s'adressait au personnel d'organismes de gestion des cours d'eau ainsi qu'aux conseillers. Le but visé était d'étudier la méthodologie des applications et de présenter des techniques éprouvées pour résoudre des problèmes pratiques. Les buts du programme abrégé sont donc complémentaires à ceux de l'Atelier.

L'Atelier s'est fixé comme but de prévoir autant de temps à la présentation des exposés qu'aux discussions subséquentes; il est important d'accorder aux discussions une part aussi importante qu'aux exposés. Je me permets donc d'inviter les participants à profiter de cette période pour apporter leur contribution à l'Atelier pour le bénéfice de leurs collègues.

Le 3e Atelier sur l'hydraulique de la glace des cours d'eau a été organisé conjointement par:

le Sous-comité du C.N.R.C. de l'hydraulique des cours d'eau

le Sous-comité du N.-B. sur la glace des cours d'eau, et

la University of New Brunswick

Je profite également de l'occasion pour vous inviter à visiter notre beau campus après les travaux de l'Atelier; je vous invite également à voir les établissements culturels et les attraits touristiques de Fredericton et, bien sûr, à voyager et à profiter de votre séjour dans notre Province.

Le Président du Comité organisateur,
K.S. Davar

PART ONE

TECHNICAL PAPERS

PREMIÈRE PARTIE

EXPOSÉS TECHNIQUES

NOTE

Papers were not refeered for scientific content and were accepted in either of the Official Languages of Canada and New Brunswick.

REMARQUE

Le contenu des documents présentés n'a pas été vérifié. Les documents ont été acceptées dans les deux langues officielles du Canada et du Nouveau-Brunswick.

SESSION A

ESTUAIRIES

Chairman: Dr. N. Elhadi, Environment New Brunswick

SESSION A

ESTUAIRES

Président: Dr. N. Elhadi; Environnement Nouveau-Brunswick

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU-BRUNSWICK

WINTER ICE REGIME IN THE TIDAL ESTUARIES OF THE NORTHEASTERN PORTION OF THE BAY OF FUNDY, N.B.

Con Desplanque[1] and Dale I. Bray[2]

ABSTRACT

As more engineering works are constructed in the upper sections of tidal estuaries in northern regions, it is imperative that engineers become aware of the primary processes related to ice formation, ice degradation and ice breakup in such environments.

Since the tidal behaviour can be estimated from astronomical considerations, it is possible to make reasonable estimates concerning the timing and the relative significance of the ice buildup in a particular estuary.

A classification system for the ice regime of well-mixed estuaries with a large tidal range is presented. Five distinct zones within the estuary are identified by considering the primary ice related processes in such estuaries.

The worst case for the formation of significant ice accumulations in these estuaries is considered to be that associated with a sequence of freezing temperatures which occurs when the difference between the neap tide and the subsequent spring tide is greatest. During this period significant vertical ice walls can form in a matter of a few days.

The ice walls which form in the upper portion of the estuaries of the Memramcook River and the Petitcodiac River can reach a height of about 5 m. As the vertical ice walls are formed they significantly reduce the normal trapezoidal cross section having side slopes in the order of 1.0V:3.5H to that of a rectangular cross section having about the same bottom width.

---

[1] Retired hydraulic engineer formerly with the Maritime Resource Management Service, Amherst, N.S.

[2] Professor of Civil Engineering, University of New Brunswick, Fredericton, N.B.

Résumé

Avec l'augmentation des travaux d'ingénierie dans les régions supérieures des estuaires des régions nordiques, il devient impératif que les ingénieurs soient conscients des procédés primaires ayant rapport à la formation, dégradation, et au bris de la glace dans ces environs.

Puisque le comportement des marées peut être évalué à l'aide de considérations astronomiques, il est possible de faire des évaluations raisonnables de l'occurrence et de l'importance relative de l'accumulation de la glace dans un estuaire particulier.

Un système de classification pour le régime de la glace d'estuaires bien amalgamés avec un grand écart de marée est présenté. Cinq régions distinctes à l'intérieur d'un estuaire sont identifiées en considérant les procédés de glace primaire dans de tels estuaires.

Le pire cas de formation d'accumulations de glace dans les estuaires est celui associé aux températures glaciales qui surviennent quand la différence entre la marée de morte-eau et la marée de vive-eau est la plus grande. Durant cette période, des murs verticaux de glace imposants peuvent se former en quelques jours.

Les murs de glace formés dans les régions supérieures des estuaires des rivières Memramcook et Petitcodiac peuvent atteindre une hauteur de 5 m. A mesure que les murs de glace se forment, ils réduisent considérablement la section normale trapézoidale, avec des pentes latérales de 1.0V:3.4H, à celle d'une section rectangulaire ayant la même largeur à la base.

L'information présentée dans ce document devrait être d'un certain intérêt pour les ingénieurs responsables de la construction d'ouvrages de contrôle et d autres ouvrages d ingénierie dans les estuaires des régions nordiques.

INTRODUCTION

McDowell and O'Connor (1977) present the following statement to define an estuary:

> "The part of the river system in which the river widens under the influence of tidal action is the estuary, but the behaviour of the estuary is influenced by the circulation of the water and solids in the sea as well as by the whole tidal part of the river system. The estuary cannot be considered in isolation; the whole system has many interdependent parts, extending from the landward limit of the tidal rivers forming it to a point offshore beyond which the effect of an individual estuary on the circulation and sediment motion can no longer be discerned."

This definition emphasizes the interaction of fresh water and sea water in the estuary but it does not explicitly consider the effect of ice in such an environment.

The purpose of this paper is to outline the winter ice regime of well-mixed estuaries which experience some of the largest tides in the world. After classifying the main features of the winter ice regime and discussing the most significant processes in such estuaries, some potential ice related problems associated with engineering works in these environments are outlined.

ICE TERMINOLOGY

There are many types of ice in the estuary and the terminology has to account for the terms used by those working with river ice and by those working with sea ice. Michel (1971) provides a good summary of river ice terminology and the World Meteorological Organization has published a glossary of sea ice terms as summarized by Dinsmore (1972).

Gordon and Desplanque (1981) discuss the various types of ice which occur in the Chignecto Bay region of the Bay of Fundy. Although their work is primarily directed toward the marine ecology of the area, it is one of the most recent publications related to ice in an estuarine environment dominated by a large tidal range.

For the purpose of this paper the following terms (arranged alphabetically) are adopted to describe ice in estuaries: Cake ice is a form of drift ice having a block-like shape with almost equidimensional major axes. The plan form is nearly circular and the bottom tends to have the shape of an elliptic paraboloid. Cake ice may be formed by the freezing of seawater on pan ice, by ice rafting (where one ice pan overrides another), or by some other process. Composite ice is a form

of drift ice of block-like shape which may attain a thickness ranging from 1 to 3 m. Hydrodynamic and wind forces can cause an ice cover of drift ice to compress and ride up on other pieces of drift ice to form the multi-layered composite ice. Drift ice is a general term to describe ice that is developed on the surface of the body of saltwater as a result of several processes and is in motion for much of the time due to the effects of the tidal currents and/or the wind. Drift ice generally consists of slush ice, pan ice, floe ice, cake ice, composite ice, refloated shorefast ice and stranded ice. The areal extent or ice cover of the drift ice is usually described as a concentration from 1/10 to 10/10. Floe ice is a type of drift ice which is formed when many pans coalesce and form a large almost circular piece of ice having a diameter greater than about 20 m but usually in the order of 50 - 100 m. Pan ice is a form of drift ice which results from the consolidation of slush ice in the freezing surface waters. A piece of pan ice is circular in shape and has relatively uniform thickness which normally does not exceed 0.15 m. The diameter of an individual pan is normally in the range of 0.5 to 10 m. Sheet ice is formed in a calm fresh water environment. It is of uniform thickness and is often characterized by a "hinge" near the banks due to the changes in water level. Shorefast ice is ice that accumulates along the shoreline of the estuary and which remains in place for most of the winter. Shorefast ice may be produced by one or more processes. At certain places in the estuary the shorefast ice will form an almost vertical ice wall. Slush ice (frazil ice) is a form of drift ice which consists of an unconsolidated mass of ice crystals. Slush ice in the estuarine environment is generally associated with the initial stages of the freezing of seawater. Stranded ice is drift ice which becomes stranded in the intertidal zone when the tide falls. Stranded ice may freeze to the substrate and become "rooted" in the sediments. If stranded ice, which is rooted, is refloated it usually transports a significant amount of bottom sediments with it.

CONDITIONS FOR THE FORMATION OF RIVER ICE AND SEA ICE

Consideration of the formation of river ice and sea ice is first given for the idealized situation where there is no significant transport of ice by river flow or by tidal currents.

In relatively still fresh water, an ice cover can develop when the surface water temperature is supercooled to about - 0.02 C. At this temperature the surface water is less dense than the deeper water and hence freezing can commence at the water surface until a continuous ice cover is formed.

The freezing point and the density for estuarine water and sea water depends upon the concentration of salt present in the water. If the salinity expressed in gm/kg of seawater exceeds 24.7 and the surface

water is cooled to the freezing point it becomes more dense at the lower temperature. In such a case the surface water sinks and is replaced by deeper, warmer water which is less dense. For the idealized case, large masses of sea water must be cooled to the freezing point before sea ice can commence to form.

In nature the conditions for the onset of ice formation are more complex than for these two idealized cases; however, it is clear that different ice formation processes are active at the land side of the estuary and at the sea side of the estuary.

In rivers, open water areas are zones where frazil ice can be formed when the air temperature is below the freezing point. In some cases, rapids or other open water areas in rivers are referred to "ice factories".

In the tidal environment, the "ice factory" may be considered to be that zone where the water is relatively shallow at low tide so that the column of water is lowered to its freezing point from top to bottom. Under such conditions floating ice crystals form at the surface of the seawater. These floating ice crystals form what is known as slush ice (or frazil ice). The slush ice typically forms pan ice due to the jostling of the slush ice by the tidal currents and by the wind.

Many pieces of pan ice coalesce to form a large almost circular piece of floe ice of relatively uniform thickness. Aerial photographs clearly show that pan ice and floe ice have slightly raised surfaces near their perimeters resulting from the impact of pieces of pan ice and/or floe ice with one another, and accumulations of slush ice washed over the edges of pan ice as it wobbles in the flow.

If the temperatures are low enough and if the tidal currents and/ or winds are not too significant, drift ice consisting of slush ice, pan ice and floe ice can congeal into a large frozen ice cover. In other cases, it is possible for an ice cover to fail and ride up over another portion of the ice cover. In this case the ice thickens and forms rafted ice. The rafted ice can break into block-like pieces of composite ice due to changes in the tidal elevations and tidal currents, and/or changes in temperature.

All of the processes described above occur at the seaward end of the estuary.

SOME IMPORTANT FEATURES OF ESTUARIES IN RELATION TO THE ICE REGIME

In a sense the estuary is the meeting place of fresh and salt water. On the landward side, the flows are in response to the transformation of

highly stochastic inputs of rainfall and snowmelt by various surface and subsurface storage and transmission processes. At the seaward side, the tidal levels and hence tidal flows and currents are in response to an essentially deterministic process which is dictated by astronomical and geographical conditions. During the winter period the flow from the land is often deterministic since the outflow is primarily from groundwater storage. At the seaward side of the estuary the freezing temperatures, which are responsible for the ice formation, and the winds, which can be partially responsible for the movement of ice, are stochastic processes superimposed on the essentially deterministic tides and tidal currents.

The estuary can be well mixed in the vertical in those cases where the volume of the tidal prism (volume between the low tide elevation and the high tide elevation) is relatively large compared to the volume of the estuary at the mean tide elevation, or the estuary can be highly stratified in those cases where the volume of the tidal prism is relatively small compared to the volume of the estuary. The ice regime is significantly different in these two environments. This paper is restricted to the case of well-mixed estuaries subjected to relatively large tidal ranges.

In such well-mixed estuaries the distribution of salinity is relatively uniform at a given cross section; however, the salinity varies from about 0 to about 35 gm/kg as one moves from the river to the ocean.

The estuary is characterized by an almost continual motion of the water in response to the river flow and the tidal regime. Ice is usually in constant motion over all or a portion of the estuary. The motion takes place in the horizontal plane and in the vertical direction. Such motion on a tide-by-tide basis needs to be considered to appreciate the types of processes which take place in this high energy environment.

DESCRIPTION OF THE STUDY AREA

The Petitcodiac, the Memramcook and the Tantramar Rivers are at the northeast end of the Bay of Fundy as shown in Figure 1. These rivers and those discharging in the Minas Basin are somewhat unique in that they are subjected to some of the largest semidurnal tides in the world.

The deterministic tide generating forces are essentially due to the motion of the moon around the earth and the earth around the sun. In addition, the ocean tides are modified by the local geographic setting. The tidal cycles of interest, the period of each cycle, and the tidal range in the upper sections of the Bay of Fundy are summarized as follows:

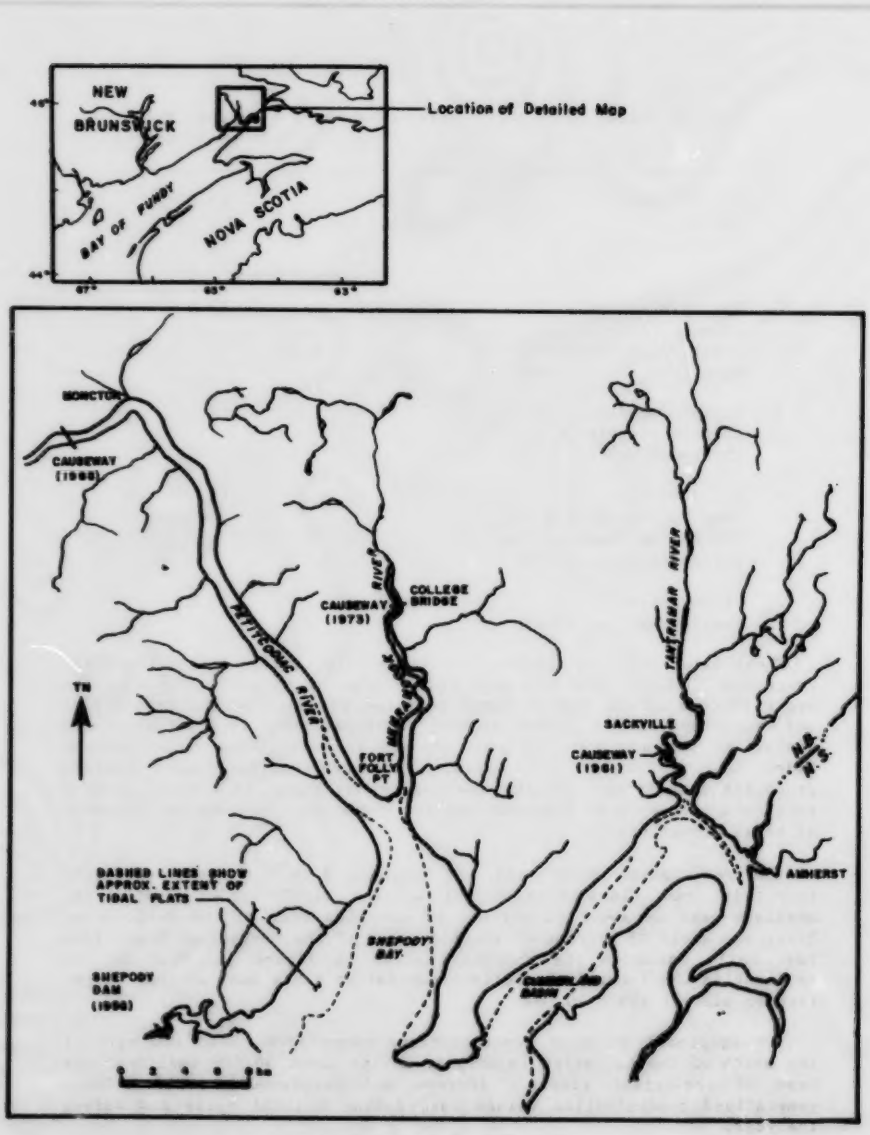


Figure 1. Location map of area at northeastern portion of the Bay of Fundy.

	Name of Cycle	Period of Cycle	Approx. Tidal Range in Metres
1.	Diurnal cycle (Primarily due to relation of moon to earth)	0.517 d (12 h 25 m)	11.0
2.	Spring/neap cycle (Due to relation of position of sun and moon relative to earth)	14.77 d	13.5
3.	Perigee/apogee cycle (Due to eccentricity of the lunar orbit)	27.55 d	14.5
4.	207 day cycle (Due to interaction of the spring/neap and the perigee/apogee cycles)	207. d	15.5
5.	Saros cycle (Last peaked in 1976)	18.03 yr	16.0

These data, based on astronomical tides only, indicate that the tidal range can deviate from the mean tidal range by as much as 5 m in the upper reaches of the Bay of Fundy near the entrance to the Petitcodiac and Memramcook Rivers. These extreme tides can occur at any time of the winter but may not occur in some winters at all. Since these extreme tides can be computed in advance, one can estimate when certain processes associated with the ice regime may occur if one is given a particular sequence of temperatures and winds and the computed sequence of tidal elevations.

The Petitcodiac River and the Memramcook Rivers have extensive intertidal mud flats even during a mean tidal cycle: Intertidal mudflats make up about 80 percent of the plan area of the Petitcodiac River and about 95 percent of the plan area of the Memramcook River from Fort Folly Point to the causeways shown in Figure 1. Most of the sediment in the intertidal mudflats consist of sizes smaller than 65 μm (median size is about 10 μm).

The suspended sediment concentrations range from about 100 mg/l at the mouth of the estuaries in Shepody Bay to about 30 000 mg/l near the head of the tidal rivers (Gordon and Desplanque, 1981). These generalized concentration values vary during a tidal cycle and during the year.

Based on the work of Gordon and Desplanque (1983), the elevation of significant tidal levels are presented in Figure 2a for areas in Cumberland Basin which is near Shepody Bay at the seaward end of the two estuaries under consideration. The important levels with respect to the ice regime for these estuaries are the spring high and low tides and the neap high and low tides. Figure 2b indicates how the time of occurrence of the highest tides varies from year to year in the Bay of Fundy.

The variation of the water level at the seaward side of the estuaries is almost sinusoidal during the tidal cycle, but is significantly distorted at the upper end of the estuary due to restricted flow into the estuary. Until a state of equilibrium is established there is a tendancy for a net upstream transport of ice. Observations near the lower end of these estuaries indicate that during high tide, the surface water continues to move upstream for about 30 minutes after the water level reaches its peak elevation.

With this background concerning the behaviour of estuaries in general and the tidal environment of the estuaries of the northeastern portion of the Bay of Fundy in particular, the primary zones associated with the ice regime are presented.

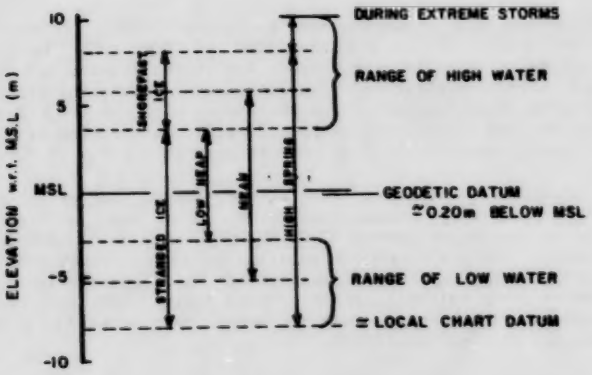
ICE ZONES WITHIN A WELL-MIXED ESTUARY

When considering the ice regime of a well mixed estuary which is subject to relatively large tidal ranges, it is possible to identify five zones which can be associated with different physical processes. The main features of each of the zones are shown schematically in Figure 3 and are outlined as follows:

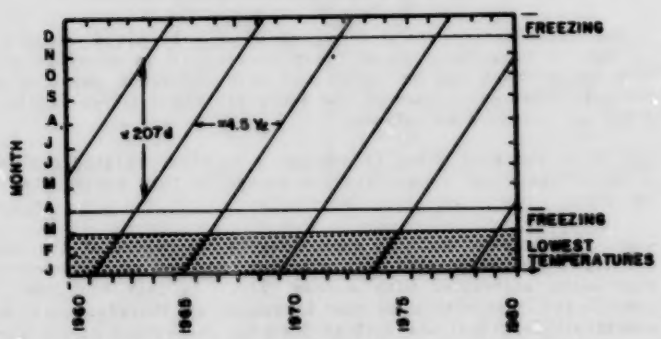
Zone 1 is the zone which is dominated by river related processes. The slope of the river is usually low enough in this region that sheet ice can form.

Zone 2 is the zone which is approximately delineated at the upper end by the high water associated with a spring tide and at the lower end by the high water associated with a neap tide. In this zone the channel is constricted appreciably by the formation of shorefast ice so that an essentially vertical ice wall is formed. The volume of the winter tidal prism is significantly reduced in this zone in comparison to the summer tidal prism.

Zone 3 is the zone which extends from the high water associated with a neap tide (lower end of Zone 2) to the low water associated with a spring tide. This is the zone which is characterized by the vast intertidal mudflats during the period of low tide. Drift ice can become stranded in this zone and in some cases the stranded ice can become frozen to the bottom of the estuary and may even become "rooted" in the



a) Range and variation of tidal amplitude in Shepody Bay, N.B. showing range over which ice types will likely dominate.



b) Generalized time of occurrence of the highest tides in the Bay of Fundy based on observed data at Saint John, N.B. Diagonal lines related to the 207 d cycle.

Figure 2. Some tidal characteristics in the Bay of Fundy which are related to ice regime in the study area.

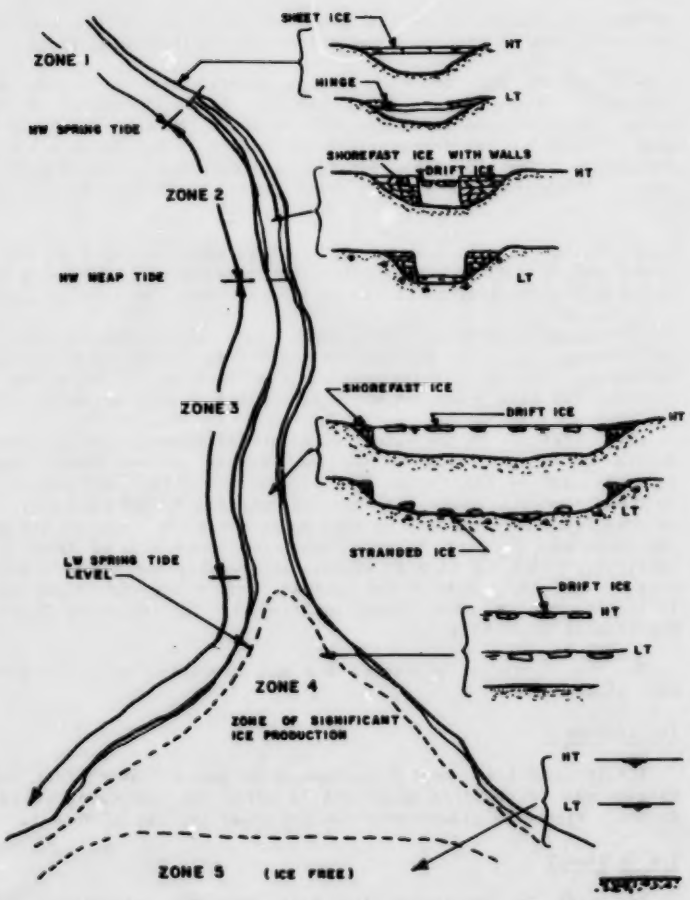


Figure 3. Schematic diagram showing ice regime zones in an idealized estuary with a large tidal range.

estuarine sediment. Stranded ice which is frozen to the bottom may be refloated when it becomes submerged during the following tides.

Zone 4 is the zone which is always covered with water but which is generally not too deep and the salinity is in the range of 10 to 25 gm/kg. In this zone it is possible to lower the temperature of a large mass of water to the freezing point. Zone 4 is the primary zone of ice production. The slush ice, pan ice, and cake ice produced in this zone can be transported into and through Zones 3 and into Zone 2 as drift ice.

Zone 5 is the zone in deep water. In this zone the depth of water is so great that it is not possible for the temperature of the mass of water to reach the freezing point and as a consequence the area is ice free.

The zones presented above imply that the boundaries are clearly defined; however, it is posible for the boundaries of the zones to change during the winter. Nevertheless, the five zones do allow one to consider the main processes which take place in such estuaries.

With reference to the estuaries of the Petitcodiac and the Memramcook Rivers as shown in Figures 1, the reaches of the rivers above the causeways can be classed in Zone 1. Since little tidal action is felt in these segments, sheet ice develops upstream of the causeways. Zone 2 extends from the causeways to some point about one third of the way from the causeways to Fort Folly Point. The lower end of Zone 3 is not explicitly known but in most winters it probably extends to a point near Fort Folly Point. Zone 4 can include Shepody Bay and Chignecto Bay as indicated by ice cover data supplied by the Atlantic Tidal Power Programming Board (1969).

The ice formation processes are now discussed and illustrated for each of the zones.

Ice in Zone 1

Ice in Zone 1 is sheet ice which often has a "hinge" near the edges in response to changing water levels after the ice cover is initially formed. Figure 4a illustrates the ice cover typical of Zone 1.

Ice in Zone 2

In order to understand the main processes associated with ice formation in Zone 2, it is important to recognize that large amounts of drift ice can be supplied to this zone from Zone 3. In addition, observations show that the ice continues to move inland for a few minutes after the high water level is reached for a given tide and as a consequence the ice does not need to strand until a short time after the occurrence of high water. Consequently, ice that is lifted to the top

a) Sheet ice in Zone 1 of the Tantramar River at Midgic Road, N.B. Some signs of tidal action are evident from the "hinge" at the sides of the channel; however, the freshwater flows dominate in this zone. (March 22, 1954 before the Tantramar River was closed by a tidal dam.)

b) Ice walls in Zone 2 of the Tantramar River at the Middle Marsh Road, N.B. The almost vertical ice walls were most likely formed in January and cake ice was deposited on top of shorefast ice during high tides in February. (March 22, 1954).

Figure 4. Illustrations of typical channel cross sections from Zone 1 and Zone 2 of the proposed classification of the ice regime for estuaries having large tidal range.

of banks or ice previously deposited on the bank will strand and be left "high and dry" when the water returns during the ebb. If this occurs during periods of freezing temperatures, the stranded ice at the higher levels is frozen solidly to the underlying base before the next high water and will not normally be refloated. If it is refloated, it will do so on a rising incoming higher tide and will usually be stranded at some point farther up the estuary.

The optimum condition for building large accumulations of shorefast ice in Zone 2 is when freezing temperatures accompany the rising tidal sequence from neap high water (LHW) to spring high water (HHW) which takes about 7 d. The sequence of ice build up is shown schematically in Figure 5 where the first drift ice is stranded just after neap high water. With each succeeding tidal cycle more drift ice is brought in to build up the shorefast ice in such a manner that almost vertical ice walls are formed. Although these walls are initially rather porous irregular shaped ice accumulations, the passing tidal water with its floating drift ice smoothens the edges and usually leaves a film of silt laden water that will freeze into a smooth surface layer of ice.

After a few days the ice free trapezoidal cross section of the channel with a side slope of about 1.0V:3.5H is transformed into a rectangular channel. As a consequence, the volume of the tidal prism and the cross sectional areas in Zone 2 are significantly reduced.

The ice walls will attain maximum height if the latter half of the 207 day spring/neap and perigee/apogee cycle happens in such a way that the period of freezing temperatures occurs from neap high water to spring high water near perigee of that particular cycle. Under such circumstances it is possible to build ice walls of the order of 5 m in Zone 2 of the estuaries of the Petitcodiac and Memramcook Rivers. Results of computations for such considerations are presented by Gordon and Desplanque (1983) and show that these extreme conditions are predictable and occur most years during periods of freezing temperatures. The summary of the results are presented in Figure 3b.

Since the foot of the ice wall must be close to the level of the lowest high water during the first tidal period, the walls on both sides of the estuary converge as one progresses inland along the estuary. At a point near the upper end of the estuary the ice walls almost touch and the tidal flow from the volume in the tidal prism is greatly reduced to the point where fresh water runoff takes over as the primary channel forming agent. This reduction in cross-sectional area can lead to flooding since the gap between the ice walls can be easily closed by fresh water ice moving downstream during the spring break-up. If this sequence of events coincides with the spring freshet; severe flooding problems can occur due to the greatly reduced channel cross sections.

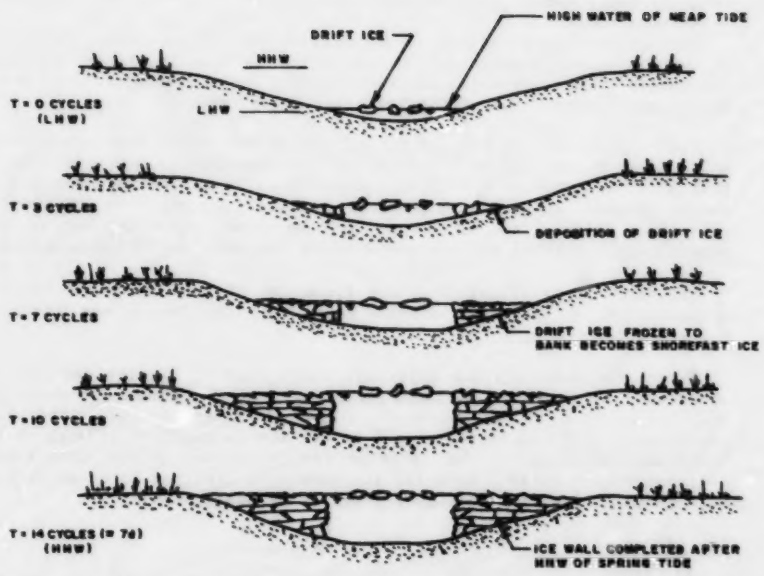


Figure 5. Schematic diagram showing the development of shorefast ice in the form of an ice wall at a cross section in Zone 2 of the estuary during periods of freezing temperatures from neap tide to spring tide.

When spring approaches, the flow of warmer fresh water and tidal water melts the bottom part of the ice walls first. This process leads to an undercutting of the ice walls which results in the deterioration of the shorefast ice in Zone 2. At places ice is sculptured into the shape of "toad stools" which take on a dark colour due to the sediments in the ice and on the ice surface. Although the thinner ice normally melts by early April, the ice walls can persist until the end of April.

Photographs are presented to illustrate some of the points presented in the discussion related to the ice regime in Zone 2. Figure 4b shows the extent of the ice walls on the Tantramar River near Sackville, N.B., before the construction of the Tantramar River causeway.

Figure 6 shows the development of ice walls on the Memramcook River at College Bridge in 1959 before the construction of the causeway at that site. Figure 6a illustrates the ice walls before the melt period commenced when the walls are about vertical at and near the bridge crossing. Figure 6b is taken at the same site and shows the deteriorated state of the ice walls after the melt season has commenced.

Figure 7a also illustrates the formation of the ice walls and the accumulation of shorefast ice in Zone 2 of the Memramcook River at College Bridge, N.B.

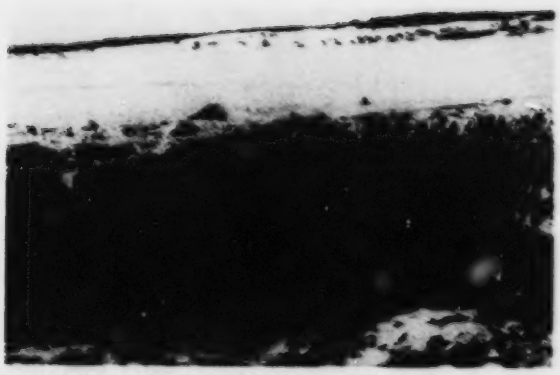
Figure 8a shows how the initially rough face of the ice wall can be smoothened by the flow of tidal water over time. Figure 8b shows the ice in Zone 2 at an advanced stage of deterioration. The high concentration of the estuarine mud in the ice is apparent as well as the "toad stool" shape of the isolated pieces of ice in the foreground. Sediment concentrations in drift ice have been reported to be as high as 0.18 gm/gm (Atlantic Tidal Power Programming Board, 1969).

Ice in Zone 3

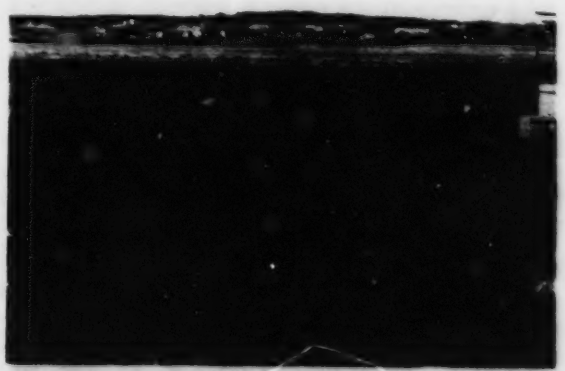
Zone 3 is characterized by the stranding of drift ice on the intertidal mudflats. Normally, shorefast ice can only develop at the edges of the estuary in the zone above the high water elevation associated with neap tide. As a consequence, the shorefast ice does not encroach on the tidal channel to any great extent in this zone.

Hinds (1875) provides a graphic description of the behaviour of ice in Zone 3 as follows:

> "During the ebb tide many of the larger blocks (of drift ice) ground on the sand base, so that when the tide is out the extensive flats are covered with ice-blocks innumerable. If the period between the ebb and the return of the flood is very cold, the stranded ice-blocks freeze to the sand-base and mud-flats and are covered by the returning tide, but only until



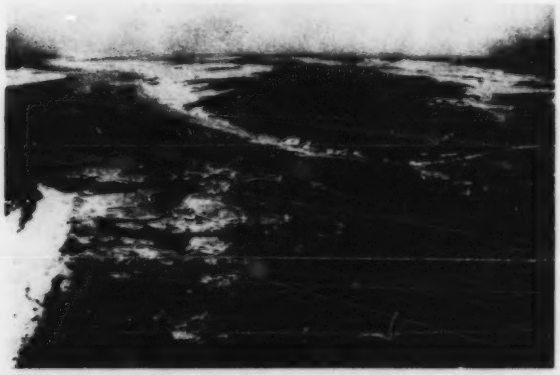
a) March 11, 1959. Almost vertical ice wall built up to marsh level. Ice wall is in the order of 3 to 4 m in height.



b) April 21, 1959. Ice wall has undergone appreciable deterioration. Ice has a relatively large amount of sediment incorporated in it and deposited on it.

Figure 6. Stages in seasonal change of the ice walls in the shorefast ice in the Memramcook River at College Bridge, N.B.

a) Development of ice walls in the shorefast ice in Zone 2 on the Memramcook River at College Bridge, N.B. Some large pieces of drift ice remain recognizable throughout the winter once the shorefast ice is formed. February 2, 1961.



b) Floe ice and pan ice during a rising tide at the confluence of the Petitcodiac River (on the left) and the Memramcook River (on the right) with Fort Folly Point near the centre of the photograph. The area is at or near the border between Zones 3 and 4. February 2, 1961.

Figure 7. Typical views showing ice formation processes in tidal estuaries having a large tidal range.

a) Smoothened surface of the ice wall in the shorefast ice. The ice wall is undercut near the water line. Cracks may result from settlement or failure of the bank. Nappan River near Nappan Station, N.S., March 31, 1959.

b) Shorefast ice after appreciable deterioration has taken place. Some deposited cake ice takes on the characteristic "toad stool" shape. Memramcook River at College bridge, April 21, 1959.

Figure 8. Some characteristics of shorefast ice at different times through the winter season in Zone 2 of the proposed classification system.

the warm tidal water succeeds in thawing the frozen sand and mud around the base of the ice-block, and it is enabled, by means of its less specific gravity to break away with a frozen layer of mud or sand attached to it. It reaches the surface of the water with a bound, and is instantly swept away by the incoming tide. The spectacle thus presented by an extensive sand-bar after a few hours of freezing weather, is most extraordinary; the whole surface of the flood or ebb becomes suddenly alive with blocks of ice, springing up from below, each carrying away its burden of sand or mud frozen to its base."

During periods of rising tide, newly generated drift ice from Zone 4 and/or refloated drift ice from Zone 3 is transported through Zone 3 to Zone 2. Zone 3 may be considered primarily as the zone for transportation of ice and for temporary storage of ice. During periods of extremely low temperatures and relatively low tides a continuous ice cover may be formed by drift ice.

Figure 7b shows drift ice near the boundary between Zone 3 and Zone 4 for the estuary of the Petitcodiac River. The relative size and somewhat circular plan form of the floe ice are illustrated in the photograph.

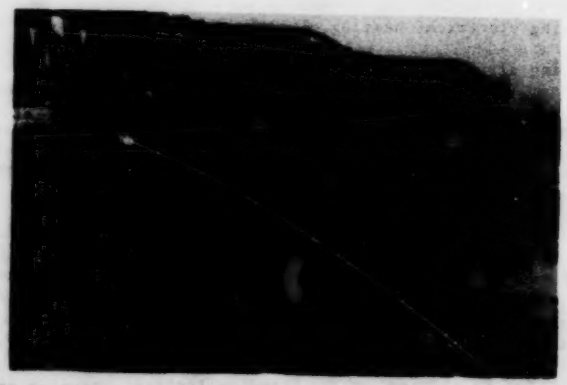
ICE RELATED PROBLEMS IN WELL-MIXED ESTUARIES WITH LARGE TIDAL RANGE

Engineers are not only interested in the knowledge of the ice regime in the estuary - but they also wish to apply that knowledge to predict the type and magnitude of problems which can be associated with civil engineering works in such an environment.

When considering the range of problems, it is clear that one has to consider the sequence of tides, the sequence of temperatures and the sequence of wind velocities. The following list outlines some of the more obvious problems which can occur in well-mixed estuaries with a large tidal range:

1. Bridge crossings can obstruct the movement of ice by the bridge piers. If the bridge is located near the border between Zones 2 and 3, it is possible that the drift ice could become frozen in place and form a continuous ice cover upstream of the bridge. If the bridge is located in Zone 2, it is possible for ice to build up on the piers by a process which is similar to that associated with the build up of the ice wall in Zone 2. As illustrated in Figure 9, this ice build up increases the effective size of the bridge piers and results in a greater reduction in the cross-sectional area at the bridge crossing.

a) March 12, 1959. Bridge piers covered with an ice coating which causes the channel to be appreciably constricted.



b) April 8, 1959. Since the thaw began on March 30, most of the ice coating on the piers has disappeared. Note the "toad stool" formations on the banks which indicate that the ice is still bonded to the banks.

Figure 9. Ice accumulation at the bridge piers of the old Tantramar River bridge near Sackville, N.B. in Zone 2 of the proposed classification system.

2. Aboiteaux which are constructed through a dyke to provide one-way drainage from the land may become inoperative if the flap gate is located in the shorefast ice. A projection can be built over the downstream side of the flap gate to prevent ice accumulation at the outlet of the aboiteaux.

3. Control works which are constructed in an estuary to control flooding of marshlands are normally associated with a causeway constructed in Zone 2 or Zone 3. The control works usually ensure that the upstream water level is controlled and as a consequence sheet ice develops there. If the structure is located in Zone 2, then the discharge capacity of the downstream channel is greatly reduced from that during the summer. When designing such control works in Zone 2 it is important to realize that the downstream channel changes from a trapezoidal channel with bottom width, b, and side slope of about 1.0V:3.5H to a rectangular channel with a bottom width of about b, where b is approximately the width at the high water of the neap tide. This point is particularly important if the design is based on open water cross sections obtained during the summer.

4. Electrically operated sliding control gates in the larger hydraulic structures in estuaries may have a build up of ice in the gate slots as shown in Figure 10. Normally such ice problems can be overcome by using electrical heating elements in the gate guides.

5. Floating structures attached to the bank of an estuary may fail due to ice related problems, particularly in Zone 3 where such structures may be adopted. In these cases it is possible for shorefast ice to develop to such an extent that the "arms" of a floating dock behave as cantilever beams rather than simple connecting links. Such a situation is illustrated schematically in Figure 11.

In addition to the specific problems associated with some engineering structures, it may be significant to evaluate the effects of the "ice jammed" estuary scenario as it affects engineering works remote from the jam site. An ice jam is most likely to occur in Zone 2 of an estuary during a period of strong landward winds, spring tides and low temperatures. During such a situation the drift ice may form a complete blockage of the channel between the ice walls. If the period of reducing tides is accompanied by freezing temperatures, the ice jam may become firmly attached in place. During the next spring tides the "ice jam" may be outflanked, and given the correct conditions, a new channel may develop around the "ice jam".

Figure 10. Ice accumulation in the 0.46 m wide stop log guide on the downstream side of the Shepody River sluice. January 8, 1959.

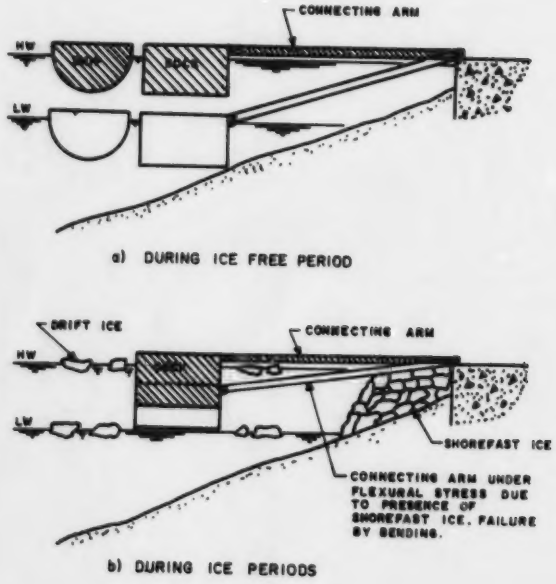


Figure 11. Schematic diagram showing the possible effects of the buildup of shorefast ice on a floating dock designed for ice free conditions in an estuary.

SUMMARY

A classification of the ice regime for a well-mixed estuary having a large tidal range is proposed as follows: Zone 1 is where freshwater sheet ice dominates; Zone 2 is where drift ice accumulates; Zone 3 is where drift ice is transported or held in temporary storage as stranded ice; Zone 4 is where most of the drift ice is formed; and Zone 5 is the area where no ice forms. The significant processes which ocur in each of the five zones have been outlined and are illustrated with photographs from the estuaries of the northeastern portion of the Bay of Fundy.

The significance of the development of ice walls in Zone 2 on the discharge capacity of the tidal channel has been pointed out as well as other factors which may affect engineering works in such an estuarine environment.

ACKNOWLEDGEMENTS

Some photographs presented in this paper are the property of the Government of Canada and were taken by Mr. C.A. Banks (Figure 7b and Figure 10) and Mr. D.A. Black (Figure 7a). We gratefully acknowledge permission to reproduce the photographs in the paper. All other photographs were taken by the senior author and are from his private collection. Some assistance was provided through an NSERC operating grant awarded to the second author. Line drawings were kindly produced through the services of A.D.I. Ltd., Fredericton, N.B. Finally, the second author wishes to thank Mr. D.P. DeMerchant for first introducing him to ice problems in tidal estuaries.

REFERENCES

1. Atlantic Tidal Power Programming Board. 1969. Feasibility of tidal power development in the Bay of Fundy. Committee Report, Halifax, N.S. Appendix 3, 20 p.

2. Desplanque, C. 1967. Ice conditions in tidal rivers and estuaries. Unpublished report of Rural Development Branch, (now Maritime Resource Management Service) 74 p. (available from MRMS, Amherst, N.S., B4H 3Z5).

3. Dinsmore, R.P. 1972. Ice and its drift into the North Atlantic Ocean, International Commission for the Northwest Atlantic Fisheries, Special Publication No. 8, pp 89-127.

4. Gordon, D.C. and Desplanque, C. 1981. Ice dynamics in the Chignecto Bay region of the Bay of Fundy, Proceedings of the Workshop on

Ice Action on Shores, Rimouski, Quebec. Sponsored by the Associate Committee for Research on Shoreline Erosion and Sedimentation, National Research Council of Canada, Ottawa, Ont., pp. 35-52.

5. Gordon, D.C. and Desplanque, C. 1983. Dynamic and environmental effects of ice in the Cumberland Basin of the Bay of Fundy. Canadian Journal of Fisheries and Aquatic Science, Vol 40, No. 9, pp 1331-1342.

6. Hinds, H.Y. 1875. The ice phenomena and the tides of the Bay of Fundy, The Canadian Monthly and National Review, Vol 8, No. 3, pp. 189-203.

7. McDowell, D.M. and O'Connor, B.A. 1977. Hydraulic behaviour of estuaries. Macmillan Press, London, 292 p.

8. Michel, B. 1971. Winter regime of rivers and lakes, Cold Region Research and Engineering Laboratory, Monograph III-B1a, Hanover, New Hampsire, 130 p.

9. Sweet, C.E. 1967. Preliminary report on ice conditions in Minas Basin, Chignecto Bay and the Annapolis Basin area of the Bay of Fundy. Unpublished report submitted to the Atlantic Tidal Power Programming Board, 22 p.

DISCUSSION

Maurice Miron

During breakup, do you detect movement of walls in Zone 2, or do they melt in place?

Reply

The shorefast ice in Zone 2 tends to melt in place. Figures 6b, 8b and 9b show that the individual blocks of ice tend to be "cemented" to the mass of shorefast ice to give the "toad stool" shaped pieces. In general, the shorefast ice in Zone 2 melts in place. In addition, the heavy loading of the ice on the thawing mud can result in differential settlement and cracking of the ice mass as shown in Figure 8a.

Fred Parkinson

Were you able to observe what type of ice there was in the ice wall? Was it columnar or polycrystalline? Photographs of spring breakup suggest it is polycrystalline and very resistant to melting. Or was this resistance caused by sediment?

Reply

I do not know of any observations concerning the type of ice crystal structure in the ice wall. Since it is primarily derived from drift ice, it is likely that the crystal structure is non-homogeneous in character. One of the difficulties of evaluating the crystal structure is the high concentration of sediment in the shorefast ice in Zone 2.

In some cases, the sediment provides an insulating layer over the ice.

In summary, it may be said that the authors are unaware of any studies of ice crystal structure in Zone 2 where the ice walls are formed.

Derek Foulds

(1) Are there year to year variations in location of the "ice-walled channel"?
(2) How did you locate the long time experienced ice observer, who proved to be so useful in the studies?

Reply

(1) The longitudinal location of the "ice-walled" channel should be relatively consistent from one time of occurence to another. We suggest that if ice walls occur in a particular year, they will extend from a point near neap high tide during formation to spring high tide during the period of formation.

It should be pointed out that in some years freezing temperature may not occur during periods when maximum range from neap tide to spring tide occurs.

By hindsight it appears that the extent of the ice accumulations were good indicators where silt would deposit after a causeway was built in the estuary. This is especially the case in the Petitcodiac River estuary in New Brunswick and in the Avon River estuary in Nova Scotia.

(2) My coauthor, Mr. Desplanque, has been making measurements and observations related to tidal phenomenon in the upper Bay of Fundy for the last 30 years. Several engineers from the region make use of his experience and knowledge of the ice regime in the area. Most recently, he has worked with biologists to evaluate the effect of ice on the benthic communities. See references by Gordon and Desplanque (1981 and 1983).

D. Andres

What effects do the ice accumulations have on water levels of floods which originate from the upstream basin?

Reply

In most of these estuaries there has been minor flooding problems which may be particularly attributed to restricted flows in Zone 2.

During a sudden mid-winter thaw and consequent heavy runoff the estuarine ice can definitely cause flooding

conditions in the upper reaches of the estuaries. This was clearly evident in the Salmon River estuary near Truro, N.S. in January 1935, March 1936, February 1948, January 1952, January 1956, February 1971, and February 1974. (Reference: Kindervater, A.D., (1975) "Flooding Events in Nova Scotia/A Historical Perspective", Inland Water Directorate, Dartmouth, N.S.)

Normally there is minimal damage due to freshwater flooding of the marshes except in those few cases where some development has taken place on the marshlands adjacent to the estuary.

As shown in Figure 1, causeways with control structures have been built on most of the estuaries. The width of the walls downstream of the control structure is about the width of one gate. If the gates are not operated properly, it may not be possible to pass the rated flows through the gates due to the excessive resistance to flow in the downstream narrow ice-walled channel of Zone 2.

The construction of causeways and dams in the upper reaches of these estuaries change the regime upstream of the structures. A fresh water ice sheet of limited thickness is formed in this region, but no large amounts of shorefast ice can form in this new environment. The resevoir formed upstream of the structure is better able to receive the river ice. When a flood raises the locally formed sheet of ice it stays in place.

Pierre R. Tremblay

Is the reduction of river cross-section due to the ice walls in Zone 2 responsible for flooding during spring flood time?

Reply

This is similar to Mr. Andres' question which is answered above. In addition one can develop a scenario for the possibility of flooding from the sea side.

If ice is blown up the estuary by high winds during a high tide when freezing temperatures occur, it may be possible to "jam" the estuary in Zone 2. The "jam" may be frozen in place during the declining tides. During the next series of spring tides, the jam may be by-passed by the tidal waters and a new channel may develop around the jam.

This sequence of events has not been documented as far as the authors know, but it may be a possible design scenario.

U. S. Panu

(1) The analysis presented in the paper is very useful in understanding the ice regime in tidal estuaries. Was there an attempt made to associate the probability of occurrence of the uppermost location of the Zone 1?

(2) Likewise, were the locations of rest of the zones discussed in the paper analyzed? A probability value provided with location and occurrence of such zones will be useful in planning and development activities along the banks (shores) of such estuaries.

Reply

(1) The uppermost location of Zone 1 is somewhere near the spring high tide during the formation of the ice walls. This location can vary from year to year depending on the sequence of tides which occur during the period of lowest temperature.

(2) No probability value was associated with the exact location of the bounds between the zones. Limited data are available. We believe that the zones as presented in this paper are somewhat conceptual in nature; however, we submit that they can be more or less defined according to significant processes in the environment under study.

Robert Gerard

This seems to have been a strangely neglected subject in the ice engineering literature and it is interesting indeed to come across one at last.

It would seem the situation in Zones 1 or 2 should have much in common with that downstream of a peaking hydro-plant. Would you expect the ice formations to be similar?

What would be the major differences (other than salinity)?

Reply

The ice walls are somewhat similar in shape, however, the supply of ice to form the shorefast ice is different. The ice in the case of the peaking plant must be from upstream whereas the source of ice for the ice walls in Zone 2 of the estuary is from downstream in Zones 3 and 4.

In addition, the ice in the estuary is forced into the ever narrowing cross sections by a sort of ratchet movement; it is forced in but cannot leave.

Bruce Pratte

The formation of the vertical walls of ice in Zone 2 is probably a good feature hydraulically since the smooth sidewalls resist further narrowing of the channel, and increase the net ebb and river flushing currents, increase the surface slope seaward, pushing the salinity intrusion seaward, resisting somewhat the further massive influx of Zone 4 ice. The tidal range, slopes, and currents would be most interesting to study, as an equilibrium configuration should eventually occur.

Also is it conceivable that the tidal range could be amplified in certain estuaries by narrowing of the channel due to ice wall buildup, thus changing the resonance characteristics?

Reply

I am unaware of any detailed studies which have been carried out to determine the effect of the modified channel in Zone 2 on the resonance characteristics in the estuary.

Since the actual volumes of the tidal prisms in Zone 2 are relatively small compared to the volumes in the upper Bay of Fundy, it is unlikely that the tidal regime is changed over the season due to the presence of the ice walls.

A simplistic argument may be developed to indicate that the tidal range should decrease during the periods in which the ice occupies a significant portion of the tidal prism in the upper reaches of the estuary. One may assume that the increase in the tidal range at these places is due to the transformation of the kinetic energy

of the flowing water into the potential energy of the raised water. In the relatively clean ice-free tidal channels not much kinetic energy is lost due to the boundary resistance; however, during the winter the shorefast ice should generally result in more resistance to flow and hence a reduction in kinetic energy.

Perhaps this problem could be pursued further by a researcher in the field of computational hydraulics.

B. Michel

It is extremely interesting to see a first paper giving the general processes of ice formation in tidal estuaries. What is described as the formation of ice walls is very much the same as what is observed in the flats of the St. Lawrence estuary. There might be one difference only, in that, the thick ice in the flats would not normally melt in flow but be transported by the flood tidal waves at spring times. The sediment content of this tidal flat ice (called batture ice in Quebec) is about the same as that given in this paper and the movement of the ice is a major cause of sediment transport in the St. Lawrence estuary.

Reply

The region described by Dr. Michel would compare more to our proposed Zone 3. Ice in Zone 3 does not necessarily melt in place, it can be moved some distance along the tidal flats. Gordon and Desplanque (1983) present discussions and photographic evidence of ice movement in Zone 3.

It should be emphasized that the concentration of sediment in the ice is a maximum obtained from a few samples. There are obvious difficulties of accurately sampling sediment in ice in such a high energy environment. The value of 0.18 gm/gm is presented to show that large amounts of sediment can be incorporated in the ice in these environments.

As a qualitative measure of the large amount of silt in the ice, it has been observed in those cases where silt-laden ice can become stranded on a marsh that substantial mounds of sediment covered the vegetation of the previous year.

Observations indicate that there is a difference between ice which is stranded on mud and ice which is stranded on a cobble beach. In the first case the adhesion to the soil is much stronger, giving the ice less opportunity to be refloated with the next tide when compared to that for the case of ice stranded on a pebble beach. This could be a reason that rock-like material is more apt to be ice rafted than estuarine muds.

Finally, the following references may be of assistance to those working in the estuaries of Atlantic Canada:

Bancroft, J. A. 1908. Ice-borne sediments in Minas Basin, N. S. Proceedings and Transactions, Nova Scotian Institute of Science, Vol. XI, pp. 158-162.

Johnson, D. 1925. The New England-Acadian Shoreline, Halifax, New York (1967 Reprint of the 1925 edition). 608 pp.

SESSION B

BREAKUP

Chairman: Dr. N. Elhadi, Environment New Brunswick

SESSION B

DEBACLE

Président: Dr. N. Elhadi, Environnement Nouveau-Brunswick

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

STUDY OF RIVER ICE BREAKUP USING HYDROMETRIC STATION RECORDS

S. Beltaos[1]

ABSTRACT

The possibility of using hydrometric station records to extract information related to ice breakup forecasting is explored. Methods for interpretation of the records are outlined and utilized to study breakup characteristics of the Nashwaak River at Durham Bridge, N.B. The results are then compared with recent insitu observations of ice conditions. It is concluded that useful but incomplete information can be extracted from existing records and a need for a theoretical framework of breakup processes is demonstrated. The value of records would be enhanced by collection of additional data such as actual ice thickness; one or more discharge measurements during breakup; and wider utilization of local observers for descriptions of ice conditions.

RESUME

Les auteurs se sont penchés sur les possibilités d'utiliser les relevés de station hydrométrique pour extraire des données liées à la prévision du déglacement. Ils décrivent leurs méthodes d'interprétation des relevés et ils s'en servent pour étudier les caractéristiques de la débâcle de la Nashwaak à Durham Bridge (N.-B.). Les résultats sont ensuite comparés à de récentes observations sur place des conditions glacielles. Ils en concluent qu'une information utile mais incomplète peut être tirée des relevés disponibles et ils démontrent qu'il serait nécessaire d'élaborer un cadre théorique des processus de déglacement. La valeur des relevés serait augmentée par la collecte de données supplémentaires comme celles qui ont trait à l'épaisseur de la glace, une ou plusieurs mesures du débit pendant la dislocation et finalement, un plus large recours aux observateurs locaux pour la description des conditions glacielles.

[1] Environmental Hydraulics Section, Hydraulics Division, National Water Research Institute, Burlington, Ontario

INTRODUCTION

During the summer 1980 meeting of the N.B. Subcommittee on River Ice (formerly: Ad Hoc Committee on Ice and Ice Jams), a question arose as to whether existing hydrometric station records could be utilized to forecast the onset and severity of river ice breakup. To explore this possibility a joint (NWRI/WSC*) study was initiated for the hydrometric station located on the Nashwaak River at Durham Bridge in New Brunswick. The undertaking of this task was facilitated by the fact that a similar study had been initiated by the writer in late 1979 for Ontario rivers, in co-operation with the WSC Guelph office.

Preliminary results of the Nashwaak River study (Beltaos and Lane 1982) indicated that useful, though incomplete, information can be extracted from existing records. This finding prompted the writer to extend the study to include factors not previously considered and assess the resulting forecast methods using insitu ice observations that have since been performed under the auspices of the N.B. Subcommittee and N.B. Environment. The results to date are reported herein.

RIVER ICE BREAKUP

When an ice-covered river basin is subjected to mild weather, two processes generally begin: increased runoff due to rainfall or snowmelt or both; and heat input to the ice cover. The former process results in increased uplift and frictional forces applied on the ice cover; and in increased water stage which, in turn reduces the support provided to the ice cover by the channel banks and provides increased channel width for movement of the cover. Heat input to the ice cover results in reduced dimensions and strength. It follows that during the mild weather spell, the forces applied on the ice cover increase while the cover's ability to resist these forces decreases. If the mild weather lasts for a sufficient time, the ice cover begins to break up which is often followed by formation of large ice jams, major ice runs and eventual clearance of the ice from the reach of interest. This general description of the breakup process includes two extreme cases, i.e., the "premature" and "overmature" breakup (Deslauriers 1968). Premature breakup occurs under conditions of intense runoff with little, if any, deterioration of the ice cover. Clearly, this type of event has the greatest damage potential, other things being equal. On the other hand, conditions of slow or no runoff with intense ice deterioration lead to overmature breakup. This event is characterized by gradual ice disintegration and has minimal potential for damage.

The first question a forecaster might ask would be how to predict whether and when breakup will be initiated. And once initiated, how

*NWRI = National Water Research Institute
WSC = Water Survey of Canada

severe it is likely to be in terms of magnitude and duration of ice jam stages at various locations.

Concerning breakup initiation, pertinent literature often advocates use of the corresponding water stage, H_B (= height above an arbitrary datum, e.g. gauge height) as a convenient and meaningful index (Shulyakovskii, 1963; Gerard, 1979; Beltaos, 1981, 1982). From our earlier discussion, it would appear that H_B is indeed a desirable parameter because it reflects the ice driving forces as well as the water surface width available for ice movement. Moreover, the above noted literature suggests that, in a given reach, H_B depends on: the thickness of the ice just prior to breakup, h_i; the degree of ice strength reduction caused by thermal effects; and the stage during freeze up when a stable ice cover forms, H_F. The latter is an index of the width of the ice cover and, excepting mature breakup events, has to be exceeded before contact of the ice with channel boundaries is eliminated. As will be discussed later, approximate values of these parameters can be extracted from gauge records. As for ice strength, there is no direct information. The best that can be done at present is to use a meteorological index intended to describe the effects of thermal deterioration.

With regard to the severity of breakup, one would ideally wish to predict the complete stage hydrograph during the breakup period at any given location. This appears to be too ambitious a task at present; it is thought more practical to limit the goal of the study to forecasts of the maximum stage during breakup, H_m. This stage can be easily identified on gauge recorder charts and is usually caused by a nearby ice jam. Theoretical considerations and field data (Pariset et al, 1966 Beltaos, 1983) have shown that the maximum stage that can be caused by an ice jam occurs when the jam has attained a condition of equilibrium and fully affects the site of interest. This equilibrium stage depends mainly on discharge, channel slope and width. During any one breakup period, H_m may or may not reach the equilibrium jam stage owing to one or more of the following reasons. (a) Ice jam located far downstream of the gauge site. Even if this jam attains equilibrium, the gauge site will experience a fraction of the jam's effect on stage. (b) Ice jam is located far upstream of the gauge site. The gauge site will again experience a fraction of the jam's effect on stage owing to attenuation effects during the jam's release. (c) Unstable jam that releases prior to attaining equilibrium. (d) Overbank flooding. Water and ice spread out onto the flood plain so that the jam's potential is dissipated. This case could be viewed as a particular instance of the unstable jam case. Considering that a major factor contributing to ice jam formation is the original ice cover itself, it is reasonable to expect that not only discharge but also competence of this cover may influence the value of H_m (see also later discussion).

DESCRIPTION OF DATA

The main data source has been the WSC record of gauge height versus time for the period 1965-81. Supplementary information consisted of

daily discharge data (WSC); meteorological data (Atmospheric Environment -"Monthly Records"); channel hydraulics in the vicinity of the gauge (B.Burrell, N.B. Environment); and recent ice thickness measurements (P.Tang, N.B. Environment). From these "raw" data, several parameters thought to be characteristic of the ice regime have been extracted as described below.

Maximum Stable Freeze Up Stage (H_F)

A typical but not universal configuration of the daily average stage hydrograph near the start of the ice season is sketched in Figure 1. The solid line represents the actual stage whereas the broken line gives the "effective" stage (=stage that would have occured had the flow been unaffected by ice). At a certain time which may be termed the beginning of freeze up, the actual stage starts to rise while the effective stage continues to drop. Eventually, the actual stage attains a peak and then declines. This sequence reflects the dynamic nature of ice cover formation in rivers. With the onset of cold weather, frazil ice forms and is initially transported freely. The effect of this moving ice on the water stage is small. As more and more frazil is produced, it begins to agglomerate into slush and pancakes. Eventually, the ice transport is impeded somewhere downstream of the gauge (due to border ice growth or other constricting feature) and an ice cover begins to propagate upstream. The presence of the ice cover causes a local stage increase which eventually begins to be "felt" at the gauge site. The gauge height then increases until the time when the edge of the ice cover arrives at the gauge site. Subsequently the gauge height decreases due to decreasing discharge and thermal smoothing of the underside of the cover. The peak stage (H_F) during this period is considered an important factor influencing the succeeding breakup because it defines the stage at which the ice cover is formed; the width of the cover is approximately equal to the channel width at the stage H_F. To eliminate brief peaks during which there is little time for freezing, H_F is defined as a daily average value. It is recognized that this definition of H_F only provides an index for the width of the stable ice cover and could, perhaps be improved by taking an average over a number of days after the peak. While this is a matter that should be investigated in the future, it was considered an unnecessary elaboration for the present exploratory study.

Moreover, it should be kept in mind that the above described freeze up process occurs frequently but not always due to occasional presence of complicating factors, e.g., severe flow and stage controls; incomplete ice cover; very rapid drop in discharge that suppresses occurrence of a peak on the stage hydrograph. Because of these and possibly other unforeseen difficulties, H_F should be determined in conjunction with consultations of prevailing weather conditions and (if available) local observers' reports* while keeping in mind its physical

* At many gauge sites, local observers are temporarily employed by the operating agency to provide brief descriptions of ice conditions at a specified frequency.

significance as outlined earlier. In the present study, interpretation of freeze up stage records presented little difficulty except on a few occasions where H_F determination was designated "uncertain".

"Winter" Peaks.

Occasionally, a brief thaw may occur during the winter period. If such a thaw causes sufficient runoff, the gauge record will show a peak which may or may not initiate breakup. In the latter case, the peak stage represents a lower limit for the stage required to initiate breakup at that time. The term "winter" peak is used conventionally and includes any peak that does not initiate breakup. While such peaks usually occur in the winter, there are instances where "winter" peaks occur a few days before the spring breakup.

Stage at Initiation of Breakup (H_B).

Usually,when a thaw does lead to breakup of the ice cover, the stage begins to rise from its fairly steady winter value and shortly after exhibits spikes and peaks that can only be caused by breaking or broken ice effects (Fig.2). A probable value of the stage at the initiation of breakup, H_B, may be fixed at the first significant spike* . Unfortunately, this definition is not always objective or meaningful. Only a probable range of H_B can then be determined, by considering: (a) the latest time for which it can be confidently assumed that there still was continuous ice cover; and (b) the earliest time for which broken ice effects became evident on the stage hydrograph. Difficulties may be experienced in cases of absence of spikes owing to very rapid stage increases caused by intense runoff or release of upstream ice jams; "misleading" spikes caused by discharge reductions due to upstream jam formation; or "overmature" breakup events where breakup can be initiated during constant or even decreasing stage conditions.

Because of such complications, H_B determination should utilize all supplementary information, e.g., prevailing weather conditions, local observers' reports and prior experience of local ice conditions. For the present study no overmature events were encountered, with the possible exception of the 1964-65 event which has been designated "undefinable". This circumstance compensated somewhat for the lack of local observers' reports that have proved extremely helpful in other studies (Beltaos, Unpublished Data).

---

* Initiation of breakup is defined herein as the instant when a sustained movement of the ice cover begins. When the cover is set in motion, the resistance to flow is reduced and the stage should tend to drop thus producing a spike on the stage hydrograph. Sometimes, however, the stage rise may be so steep as to suppress spike appearance. Only a slowdown in the rate of rise would then be evident.

Maximum Breakup Stage (H_m)

This is the maximum stage reached during the breakup period and its determination is straightforward (Fig. 2).

Effective Stage and Maximum Ice Effect on Stage (ΔH_m)

The ice effect on stage is the difference between the actual stage and the effective stage. The time of maximum ice effect can usually be determined by simple inspection (Fig. 2) and does not necessarily coincide with the time of H_m.

Daily average discharge values are estimated by WSC based on interpolations between discharge measurements as well as on such evidence as upstream and tributary flows, runoff and weather conditions, etc. Such estimates may involve considerable error. This has repercussions on the accuracy of the effective stage which is determined by joining daily values plotted at noon of each day. For the Nashwaak R. at Durham Bridge, very little confidence can be placed on discharge estimates during breakup conditions (Beltaos and Lane 1982).

Ice Thickness (h_i)

Ice thickness can be estimated from WSC discharge measurement notes. Such notes give the distance from the water surface to the bottom of the ice which, under free flotation conditions, represents about 92% of the total ice thickness. However, this assumption may or may not be valid depending on whether there is significant bank support of the ice or snow cover which may cause the free water surface to rise above the top of the ice. The presence of a slush deposit under the solid ice may render thickness values completely unreliable because the notes would then show the distance from the water surface to the bottom of the slush. Another source of error may be (unreported) instances when "water surface" has been used nominally, i.e., substituted by a more convenient datum such as the top of a deep snow layer.

Usually, a few ice thickness values will be available during any one winter season. These can be plotted versus time and extrapolated to the start of the mild weather spell that led to breakup. Where the winter season involves highly variable weather conditions, it may be preferable to extrapolate using a more complex correlation, e.g., h_i versus accumulated degree-days of frost. Such procedures would generally give fair indications of h_i at the time breakup starts but ignore thickness reductions that may occur during the pre-breakup period (onset of mild weather spell to onset of breakup). This assumption is considered adequate for the present in view of (a) the crudeness of the other data involved; and (b) the partial accounting of this effect by introducing a meteorological index of heat input to the ice cover.

Meteorological Index of Ice Strength

Few data on ice strength at the time of breakup are available and the manner of ice strength reduction by thermal effects is not well

understood at present (Frankenstein, 1961; Korzhavin, 1971; Butyagin, 1972). In general, it is reasonable to expect that ice strength will decrease with increasing amounts of heat absorbed by the ice cover but there is no consensus on the most appropriate index for the latter. A very simple and well known index is the accumulated degree-days of thaw, S_T (see for example, Williams, 1965; Bilello, 1980). However, S_T can only be satisfactory in cases where the time of year when breakup occurs and the number of "thawing" days do not vary appreciably. Otherwise, the important effect of solar radiation will not be considered. For example, a sunny day in April would be much more effective in weakening the ice than a cloudy day of the same average air temperature in January. To fully account for thermal effects on ice strength, several parameters are needed in addition to air temperature, e.g., short wave radiation, cloudiness, wind speed, water temperature, snow cover, ice composition, etc.

Unfortunately, not all of this information is usually available and even if it were, it would be impractical to attempt multiple correlations with so many parameters. Shulyakovskii (1963) suggested the use of a calculated value of heat input to the ice cover from the surface, thus ignoring heat transfer from the water since water temperature is, as a rule, unknown. A similar but somewhat simplified approach was suggested by Williams (1965). Bulatov (1972) outlined a method for computing ice strength based on theoretical and experimental correlations with radiation effects. However, Bulatov's paper was too general to permit application of his method by others. Ashton's (1983) analysis is similar to Bulatov's and shows that the main agent of deterioration is the penetrating solar radiation, once the ice has been warmed to 0°C. Additional radiation absorption causes melting at the grain boundaries with a resulting decrease in strength. However, Ashton's analysis cannot be applied to the data under consideration because information on snow cover, albedo and ice structure is lacking.

Evidently, only empirical indices of ice strength can be employed at present. Some of the simplest ones are accumulated degree-days of thaw, hours of sunshine and solar radiation but their simultaneous consideration would complicate the analysis. Shulyakovskii's (1963) single heat input parameter, $\sum q$, has the advantage of simplicity as well as a background of practical usage and was thus utilized by Beltaos and Lane (1982). However, there is no theoretical evidence that this parameter adequately accounts for the qualitatively different effects on ice strength of the various heat components involved.

ANALYSIS OF DATA

Table 1 summarizes the data for the Nashwaak R. at Durham Bridge (Fig.3). Of the 21 freeze up - breakup events that occurred during the period of record (1965 -81), one has proved impossible to interpret, while six presented serious difficulties. At the time of writing the report by Beltaos and Lane (1982), only a few ice thickness values were available and thus no attempt was made to consider h_i in the analysis. Subsequently, additional ice thickness data were made available to the

writer by P. Tang of N.B. Environment which enabled determination of h_i for many of the events under consideration. For events without any thickness measurements h_i was estimated via a correlation between measured values and time from H_F. This procedure involves errors as large as 30%. Water surface to bottom of ice distances quoted by WSC have been divided by 0.92 to obtain h_i though this is recognized to be a first approximation. $\sum q$ values quoted by Beltaos and Lane (1982) have been revised to account for daily variations of associated heat input coefficients but only in a few instances did this result in substantial changes.

Initiation of Breakup.

Beltaos and Lane's analysis (1982) followed Shulyakovskii (1963), after some initial verifications of the basic premises. First, H_B was plotted versus H_F where a trend for H_B to increase with H_F was indicated. However, there was considerable scatter suggesting additional effects. Next, the difference $(H_B - H_F)$ was postulated to depend on h_i and $\sum q$, the total amount of heat input to the ice cover per unit surface area. The latter is an accumulation of daily heat fluxes (q) during daylight hours until the time of breakup initiation; heat transfer from the water is ignored. Calculation of $\sum q$ involves many simplifying assumptions so that $\sum q$ must be viewed as a mere index of the true amount of absorbed heat (see Beltaos and Lane 1982 for details of the calculation). A plot of $(H_B - H_F)$ versus $\sum q$ indicated the expected trend but exhibited considerable scatter. To explore the possible effects of h_i, the following procedure was adopted. First, $(H_B - H_F)$ was plotted versus h_i by noting the value of $\sum q$ beside each data point. This indicated an increase of $(H_B - H_F)$ with h_i and a decrease with $\sum q$. The upper envelope of the data points, assumed representative of the case $\sum q = 0$, was then described by the straightline $(H_B - H_F) = 2.5\ h_i$*. Next, the deviation of any one data point from the upper envelope [$= 2.5\ h_i - (H_B - H_F)$] was computed and plotted versus $\sum q$, as shown in Fig. 4. Data ranges in Fig. 4 indicate instances where only ranges for H_B could be identified; vertical ranges indicate cases where lower and upper limits of H_B occurred within a short time period so that the corresponding $\sum q$'s were nearly equal. Data points with arrows denote winter peaks or otherwise known limits for H_B; such points are occasionally of little value (e.g., two uppermost points at $\sum q = 0$) but often give useful indications as to how a correlation line should be drawn.

Fig. 4 confirms the anticipated trends but with considerable scatter. The latter can be partly attributed to the crudeness of H_B determinations (no local observers' reports) and the empiricism introduced in the analysis (lack of a theoretical framework for breakup processes). A compensating feature is that even a large error in predicting H_B usually translates to acceptable error in forecasting

* Linear plots of this kind have also been found by the writer at other sites but with different numerical coefficients (Beltaos, unpublished data).

the time of H_B because the prevailing temporal gradients of stage are usually large.

It may be noticed in Fig. 4 that two data ranges are plotted for the 1979 event, designated (1) and (2). The former reflects the interpretation given by Beltaos and Lane (1982) and involves serious uncertainty; it plots far off the band of the other data. Later on, it was discovered (P. Tang, personal communication) that a site visit by WSC staff in March 1979 indicated the presence of intact ice cover which dictated the following revision. What was originally thought to have been breakup initiation was in fact a winter peak whereas breakup was initiated later in March. The event designated 1979(2) reflects the new interpretation and plots at a much improved position. This result illustrates the importance of local observers' reports.

Maximum Breakup Stage

As discussed earlier, flow discharge is a major factor influencing H_m. However, discharge data for the Nashwaak River study are uncertain so that the plot of Fig. 5, showing H_m' ($=H_m$ - stage at zero discharge) versus prevailing discharge is of qualitative value. It is noted that some of the data points in Fig. 5 represent conditions of maximum ice effect, ΔH_m, in instances where the latter did not occur at the same time as did H_m. Also plotted in Fig. 5 is the theoretical relationship between equilibrium jam stage and discharge for comparison (Beltaos 1983). The latter is seen to provide a satisfactory upper envelope up to a certain discharge, but to consistently overpredict the stage beyond this discharge. This is a typical trend, reflecting the fact that increasing discharge reduces the probability of equilibrium jam formation (Beltaos 1983). For practical purposes, an upper envelope of the data points could be drawn and used to forecast potential H_m values. Whether and how closely the potential H_m is to be realized in a given season depends on the number and stability of ice jams that form near the gauge site, as discussed earlier. In turn, such effects are controlled by channel and floodplain configuration as well as the competence of the ice cover during breakup. The former factor is difficult to assess at present because the behaviour of ice jams is unknown once the bankfull stage is exceeded (see also Calkins 1983). On the other hand, experience suggests that the competence of an ice cover should be an important factor influencing H_m and this possibility is considered next.

Since the competence of an ice cover can be defined in terms of its strength, thickness and width, it may be of interest to explore $\sum q$, h_i, and H_F (rough index of ice cover width) as possible factors influencing H_m. Fig. 6 shows H_m plotted versus H_F. The data points define an upper envelope that increases with H_F. The deviation of the observed value of H_m from the corresponding upper envelope value is plotted versus $\sum q$ in Figure 7. This results in another upper envelope that confirms the anticipated trend. It thus appears that H_F and $\sum q$ define a potential or, an upper limit for, H_m. Whether and how closely this potential will be realized in any one breakup event, depends on a number of other factors, e.g., discharge, local jamming

conditions, etc. One would expect that h_i should also be relevant here but, owing to discharge uncertainties, this possibility has not been investigated herein, though h_i effects on H_m have been discerned elsewhere (Beltaos, Unpublished Data). Moreover, it is noted that strictly speaking, $\sum q$ should be calculated to the time of H_m. However, the value used in Fig. 7 applies to the time of H_B. This was thought sufficient given that the present study was exploratory.

Frequency of Occurrence of H_m

A simple frequency analysis on H_m values was also performed by Beltaos and Lane (1982). The fact that, occasionally, there have been two breakup events in the same season was ignored and all events were assumed to be independent so as to increase the effective length of record. This may or may not be valid but more data are needed to clarify this point. For the present study, it was found that the above approach resulted in a frequency curve that differed very little from the one obtained by use of only the highest breakup stage in any one season.

For convenience of plotting, use has been made of, $H_m' = H_m$ - stage at zero discharge. In this manner, the event $H_m' \geq 0$ has a probability of 1. After performing the frequency analysis, H_m' can be plotted versus probability on various types of charts as a means of exploring the mathematical form of the H_m' distribution. Gerard and Karpuk (1979) suggested that the log-normal distribution is a possible candidate and found a linear relationship after plotting their data on log-normal probability paper. Figure 8 shows that only in the range $0.1 \leq P \leq 0.9$ do the present data adhere to a linear relationship.

Limitations

Clearly, the results presented so far are site-specific and empirical. Therefore, extrapolation to other sites or hydrometeorological conditions different from those covered by the years of record is not justified. Accumulation and comparison of several case studies such as the present would facilitate development of more general forecasting methods.

COMPARISON WITH OBSERVATIONS

Since 1981, ice conditions in the Nashwaak River near Durham Bridge are monitored under the auspices of the N.B. Subcommittee on River Ice and N.B. Environment. The results of the field observations are used in this section to assess the effectiveness of the relationships derived so far.

1981-82 Event

Ice effects on stage commenced on Dec. 26, 1981, and a value of 2.18 m was chosen for H_F on Dec. 28. Breakup was initiated at about 1100 h on Apr. 2, 1981, with H_B = 2.50 m and H_m = 3.00 m occurring

at 1800 h on Apr. 3. From measurements, h_i was estimated as 0.61 m and $\sum q$ was calculated as 5784 J/cm^2. The quantity 2.5 h_i - (H_B - H_F) is 1.21 m and inspection of Fig. 4 indicates that the data point for this event would not fit the trend defined by the historical data. To explain this discrepancy, a close examination of the weather records was undertaken and revealed a highly atypical sequence of events: A warming trend began on Mar. 11 and continued until Mar. 20. Subsequently, the weather turned cold but q values remained positive, excepting the dates Mar. 22, 27, 28 and 29. A total of 15 cm of snow fell during the period Mar. 19-22. A continuous warm trend began on Mar. 30 and led to breakup. Between Mar. 11 and 29, a net of 34.3°C - days of frost was accumulated. This sequence of events suggests that sustained thermal ice deterioration would have started on Mar. 30 even though the value of $\sum q$ up to Mar. 29 was 3789 J/cm^2. This illustrates a shortcoming of Shulyakovskii's $\sum q$ calculation. The latter only accounts for heat exchange during daylight hours and would thus seriously underestimate recovery of ice strength during a cold spell that intervenes between two warm ones. If $\sum q$ were accumulated from Mar. 30 on, a value of 1995 J/cm^2 would be obtained. This would improve the plotting position of the 1981-82 event in Fig. 4. However, such a correction involves a measure of arbitrariness and the writer cannot see how to improve this situation without resort to a theoretical model of ice deterioration. Though some research has been done in this regard (Bulatov 1972; Ashton 1983), it has not advanced to the point where it can be applied in practice. For the present, it can only be hoped that the forecaster would recognize atypical events and make necessary allowances based on experience.

With $\sum q$ = 5784, Fig. 7 indicates that the quantity (H_m - 1.22 - 1.18H_F) should not exceed -0.68 m which gives H_m < 3.11 m, as was the case (H_m = 3.00 m). If $\sum q$ were taken as 1995 J/cm^2, Fig. 7 would have given H_m < 3.63 m. In cases where reliable discharge data are available, a plot such as Fig. 5 could also be used to improve forecasts of the potential H_m value. However, this is not possible in the present study owing to the serious uncertainties associated with breakup discharges.

1982-83 Events

Ice effects on stage commenced on Dec. 13, 1982, while a value of 2.50 m was chosen for H_F on Dec. 19. A mild weather spell in January led to breakup with H_B = 2.65 m at 0900 h on Jan. 12 and H_m = 3.83 m at 1500 h on Jan. 12. The values of h_i and $\sum q$ are estimated as 0.24 m and 375 J/cm^2 respectively. It follows that 2.5 h_i - (H_B - H_F) = 0.45 m and this would plot satisfactorily in Fig. 4. Use of Fig. 7 gives H_m < 4.24 m, as was the case (H_m = 3.83 m).

The peak stage during the January event was caused by a local ice jam that did not release but froze in place as cold weather resumed. A new H_F of 2.55 m occurred on Jan. 17. Breakup was initiated at 1000 h on Mar. 22 with H_B = 3.30 m, $\sum q$ = 3669 J/cm^2 and H_m = 3.97 m at 1800 h on Mar. 22. No ice thickness measurements are available for this event, hence h_i can only be estimated, as follows. If other years'

experience is used and the presence of the frozen jam is ignored, h_i would be estimated as 0.61 m. On the other hand, the thickness of the jam at the time it formed is estimated to have been about 1.2 m, (see for example, Beltaos 1983). Calkins (1979) has shown that ice growth is accelerated in the presence of a porous ice accumulation under the lower boundary of a solid ice cover. If, as a first approximation, h_i is assumed to increase as the square-root of degree-days of frost, then a factor of $\sqrt{1/p}$ should be applied to the normally expected ice thickness (p = porosity). For p = 0.4, this gives $h_i = 0.61/\sqrt{0.4} = 0.96$ m. With this, the value of $2.5\ h_i - (H_B - H_F)$ becomes 1.66 m which would plot satisfactorily in Fig. 4. Use of Fig. 7 gives $H_m < 3.83$ m. The observed H_m was 3.97 m, i.e., 0.14 m higher than would have been thought possible from the historical data. This is very likely due to the extremely thick ice cover caused by the freezing of the January jam.

CONCLUSIONS

The present results indicate that useful though incomplete information can be extracted from existing gauge records. This information can be utilized in forecasting the onset and potential severity of breakup, subject to the limitations outlined next.

The present analysis is empirical and site-specific; hence, it cannot be extrapolated to other sites or to hydrometeorological conditions that are not covered by the years of record. While studies similar to the present can be used as an aid to forecasting, it was shown that some reliance on experience would be necessary for unusual events. The lack of a theoretical framework for breakup processes is considered a major obstacle to eliminating empiricism from pertinent forecasting methods. Accumulation and comparison of additional case studies would contribute toward this goal.

As a by-product of this study, several instances were identified where moderate increases of the gauge operation effort would greatly increase the value of records for breakup-related studies. These include measurement of the true ice thickness and, where applicable, delineation between solid and slush ice layers; wider utilization of local observers and increase of reporting frequency during freezeup and breakup; and performing one or more discharge measurements during breakup events.

REFERENCES

ASHTON, G.D., 1983, First-Generation Model of Ice Deterioration, Proc., Conference on Frontiers in Hydraulic Engineering, ASCE, Cambridge, Mass. pp. 273-278.

BELTAOS, S., 1981, Ice Freeze Up and Breakup in the Lower Thames River: 1979-80 Observations, National Water Research Institute, Unpublished Report.

BELTAOS, S., 1982, Initiation of River Ice Breakup, Proc., Fourth Northern Research Basin Symposium Workshop, Norway, pp. 163-177.

BELTAOS, S., 1983, River Ice Jams: Theory, Case Studies and Applications, J. of Hydraulic Engineering, ASCE, Vol. 109, No. 10, pp. 1338-1359.
BELTAOS, S, and LANE, R., 1982, Ice Breakup Characteristics of the Nashwaak River at Durham Bridge, N.B., National Water Research Institute, Unpublished Report.
BILELLO, M.A., 1980, Maximum Thickness and Subsequent Decay of Lake River and Fast Sea Ice in Canada and Alaska, U.S. Army CRREL Report 80-6.
BULATOV, S.N., 1972, Computation of the Strength of the Melting Ice Cover of Rivers and Reservoirs and Forecasting of the Time of Its Erosion, Proc., IAHS Symposium on the Role of Snow and Ice in Hydrology, Vol. 1, Banff, IAHS-AISH Publication No. 107, pp. 575-581.
BUTYAGIN, I.P., 1972, Strength of Ice and Ice Cover (Nature Research on the Rivers of Siberia), U.S. Army CRREL, Draft Translation 327.
CALKINS, D.J., 1979, Accelerated Ice Growth in Rivers, U.S. Army CRREL Report 79-14.
CALKINS, D.J., 1983, Ice Jams in Shallow Rivers with Floodplain Flow, Canadian J. of Civil Engineering, Vol. 10, No. 3, pp. 538-548.
DESLAURIERS, C.E., 1968. Ice Breakup in Rivers, Proc., Conference on Ice Pressures Against Structures, NRC Technical Memorandum No. 92, pp. 217-279.
FRANKENSTEIN, G.E., 1961, Strength Data on Lake Ice, U.S. Army SIPRE Technical Report 80.
GERARD, R., 1979, River Ice in Hydrotechnicl Engineering, A Review of Selected Topics, Proc., Can. Hydrology Symposium 79, Vancouver, NRCC No. 17834, pp. 1-29.
GERARD, R. and KARPUK, E.W., 1979. Probability Analysis of Historical Flood Data, J. of the Hyd. Div., ASCE, Vol. 105, No. HY9, pp. 1153-1165.
KORZHAVIN, K.N., 1971. Action of Ice on Engineering Structures, U.S. Army CRREL AD 723 169.
PARISET, E., HAUSSER, R., and GAGNON, A., 1966. Formation of Ice Covers and Ice Jams in Rivers, J. of the Hyd. Div., ASCE, Vol. 92, No. HY6, pp. 1-24.
SHULYAKOVSKII, L.G. (editor), 1963, Manual of Forecasting Ice Formation for Rivers and Inland Lakes, Israel Program for Scientific Translations, Jerusalem 1966.
WILLIAMS, G.P., 1965, Correlating Freeze Up and Breakup with Weather Conditions, Can. Geot. J. Vol. II, No. 4, pp. 313-326.

Table 1. Summary of Breakup Characteristics

Season	H_F (m)	H_B (m)	H_m (m)	h_i (cm) From Measurements	h_i (cm) Estimated ±30%	$\sum q$ (J/cm^2)	Remarks
1964-65	2.56	>2.20	NA		55	2125	Breakup undefinable
1965-66	1.40	1.58	2.23		73	5420	
1966-67	1.80	1.71	1.73	70		5944	
1967-68	2.34	3.25	3.87		43	425	
-"-	3.35	>2.78	NA		52	3748	
-"-	3.35	2.71	3.40		52	6246	
1968-69	2.25	>1.84	NA		38	0	
-"-	2.25	1.87	1.87		74	4240	H_B uncertain
1969-70	3.44	1.98-5.31	5.31		41	511	H_B uncertain
1969-70	2.29	1.43-1.80	2.05	37		4782	H_B uncertain
1970-71	1.65	>2.07	NA	40		197	
-"-	1.65	>1.83	NA	64		2119	
-"-	1.65	0.90-1.31	NA	64		6819-7359	
1971-72	1.19	>1.34	NA	18		251	
-"-	1.19	>1.43	NA	50		0	
-"-	1.19	>1.59	NA	60		1370	
-"-	1.19	>1.74	NA	79		0	
-"-	1.19	1.62-2.19	2.49	79		978-1467	
1972-73	2.72	>2.79	NA		50	241	
-"-	2.72	>2.16	NA		60	3251	
-"-	2.72	1.91-2.35	2.61		60	5294	
1973-74	2.20	>2.22	NA		75	2900	
-"-	2.20	1.56-2.19	2.19		76	5353-7162	
1974-75	2.19	>2.15	NA		70	2205	
-"-	2.19	1.48-1.68	1.68		70	4642-6267	H_B uncertain
1975-76	2.44	>3.38	NA		59	338	
-"-	2.44	2.19	2.94	73		3933	
1976-77	1.89	>2.69	NA	30		0	H_F and H_B uncertain
-"-	1.89	>1.91	NA		70	3897	H_F = 2.29 m
-"-	1.89	1.49-1.53	2.29		72	7442	might be better
1977-78	1.82	2.15-2.51	2.51		25	21	
-"-	2.74	1.42-1.54	1.76	64		8176	H_F uncertain
1978-79	1.10	<1.93	2.57	30		293	
-"-	3.40	>3.26	NA	34		1074	
-"-	3.40	>3.12	NA	61		2650	
-"-	3.40	2.07-2.56	3.20	61		6234-6719	
1979-80	2.25	2.71-3.01	3.03	55		658	
1980-81	2.41	1.81-2.11	2.12	52		736-2542	

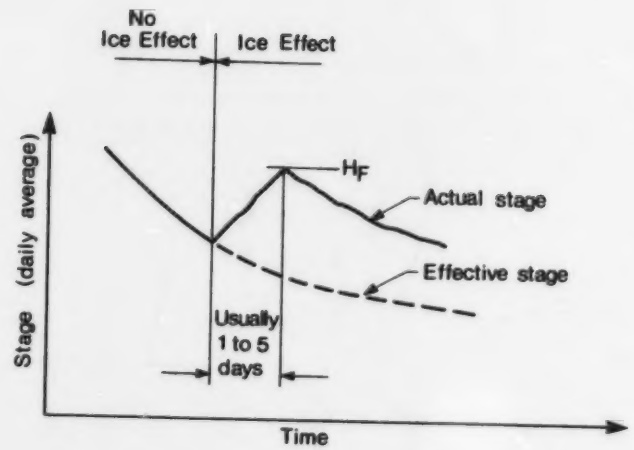


Fig. 1 Schematic illustration of daily stage variation with time during beginning of freeze up.

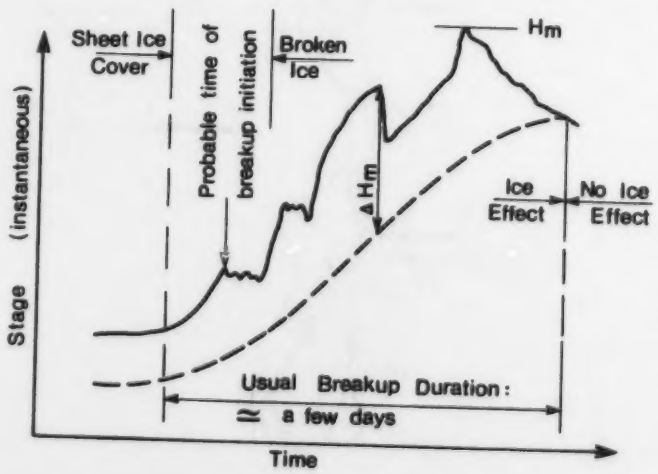


Fig. 2 Schematic illustration of instantaneous stage variation with time during breakup.

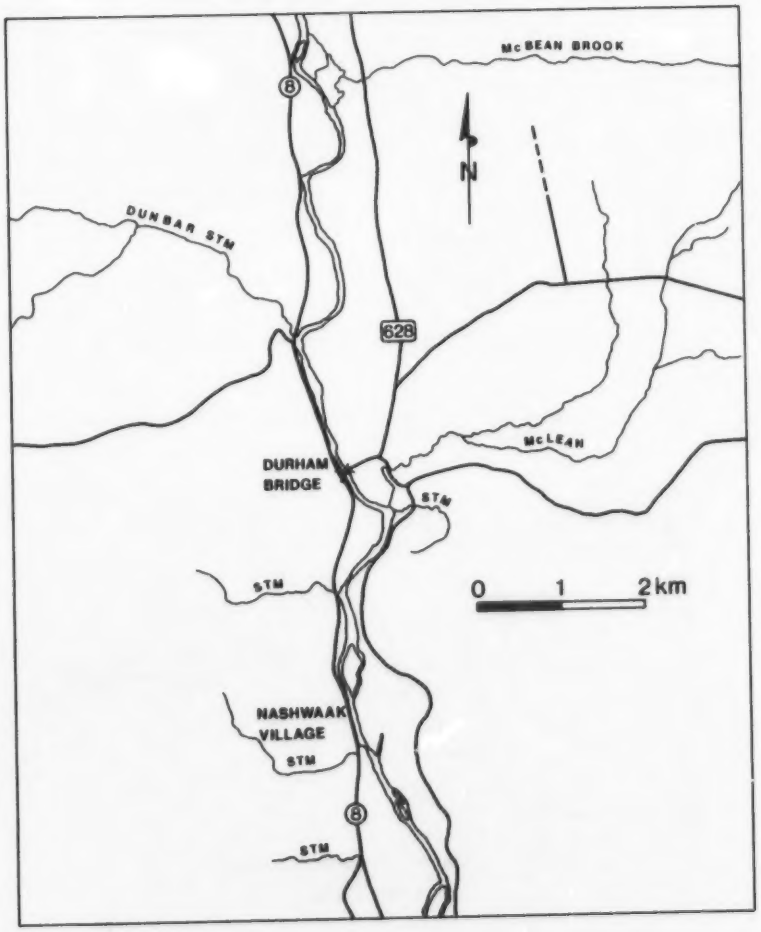


Fig. 3. Plan of Nashwaak River in the vicinity of Durham Bridge.

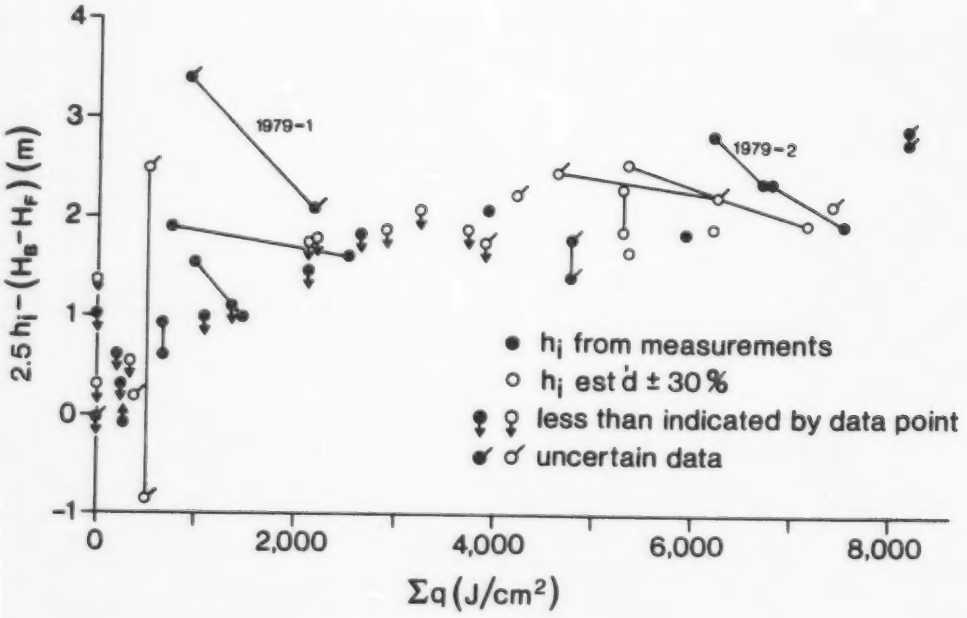


Fig. 4. Variation of $2.5h_i - (H_B - H_F)$ with Σ_q

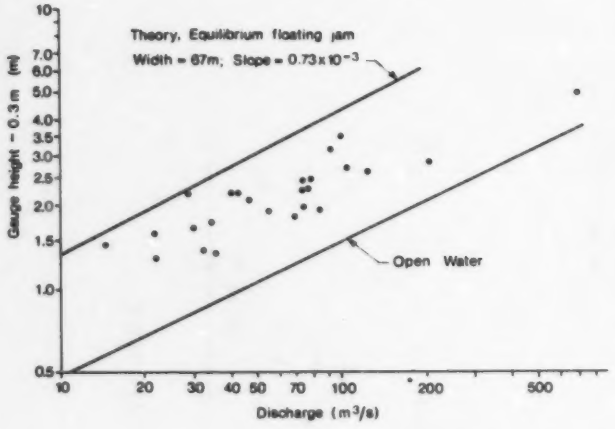


Fig. 5. Effect of discharge on breakup stage.

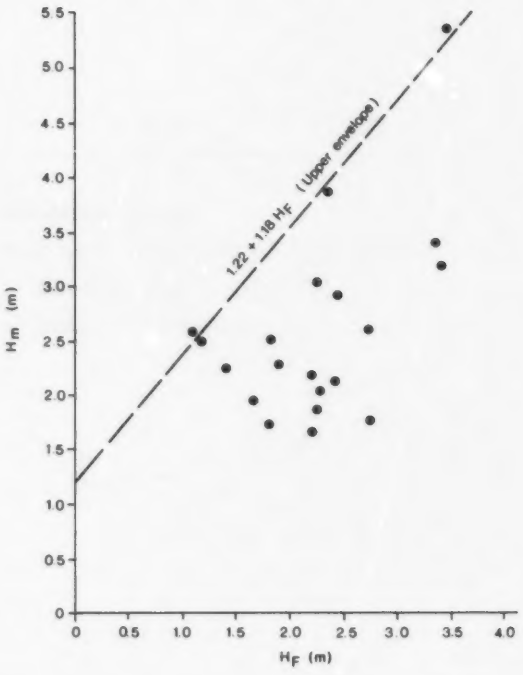


Fig. 6. Maximum stage during breakup versus H_F.

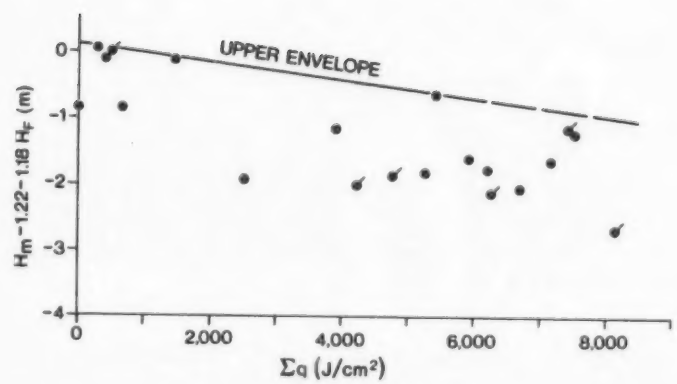


Fig. 7. Effect of Σ_q on H_m; legend same as for Fig. 4.

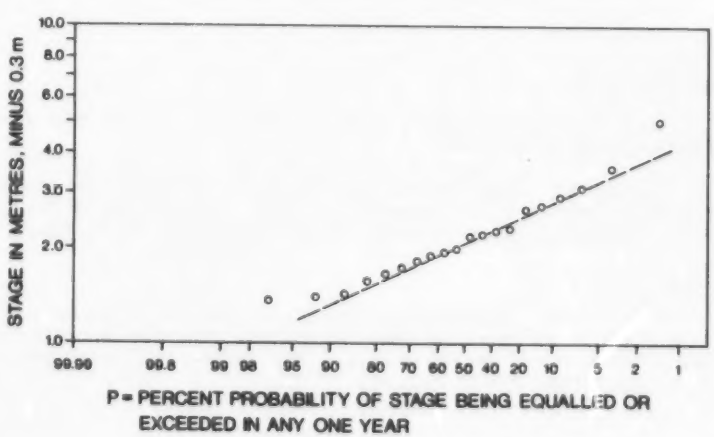


Fig. 8. Frequency curve of breakup peak stage.

DISCUSSION

Razek Abdelnour

The proposed model by Dr. Beltaos does not include the effect of water temperature. Would you think that the temperature would have an effect on the prediction of ice breakup? If yes, could that be included in your empirical model in terms of heat (Σq) gained from the bottom surface of the ice?

Reply

The water temperature should have a significant effect on breakup. However, it is not possible in the present study to determine the amount of heat transferred from the water to the ice cover because water temperature data are not available. At the same time, the writer feels that the water temperature effect is, at least partly, taken into account via the parameter Σq because the greater Σq is, the greater the heat transfer from the water would be expected to be. However, no such correlation can exist in reaches where water temperature is controlled by other than natural means, such as thermal effluents.

Maurice Miron

Was the study made with one station or multiple ice effect range stations?

Comment: some stations have no ice effect (outlet of lake) and the thickness of the ice at gauge may be up to few feet! Mean control free of ice as spillway.

Reply

The study considered one hydrometric station. In a lake or reservoir outlet situation, the ice should have little effect on water levels. The writer's study applies to uncontrolled river reaches.

Derek Foulds

Did you sort the information to pick out the years in which there were ice jam floods?

Reply

This was not done in the paper. The important factors influencing H_m were shown to be H_F, Σq and Q(= discharge); h_I should also have an effect but my analysis has not considered h_I because of the empiricism involved. To provide a more detailed response to Mr. Foulds' question, the writer has sorted out the events with the three highest and three lowest values of H_m, as shown in Table 2. The results of Table 2 can be tentatively explained by the hypothesis that h_F, Σq and h_I define a potential value of H_m which may or may not be realized, depending on runoff.

Table 2. Parameters associated with highest and lowest values of H_m.

H_m (m)	H_F (m)	h_I (cm)	Σq (J/cm)	approx.Q (m^3/s)	Season
			High H_m's		
5.31	3.44	41	511	700	1969-70
3.87	2.34	43	425	100	1967-68
3.40	3.35	52	6246	93	1967-68
			Low H_m's		
1.68	2.19	70	6267	32	1974-75
1.73	1.80	70	5944	15	1966-67
1.76	2.74	64	8176	47	1977-78

The highest value of H_m occurred under conditions of high H_F, low Σq, moderate h_I and high Q. On the other hand, the lowest H_m occurred with moderate H_F and large h_I but with low Q and high Σq. It is further noted that the high H_m's are associated with moderated h_I's whereas the low H_m's involve large h_I's. This seems to contradict intuition but it is felt that the true effect of h_I is masked by the effects of H_F, Σq and Q. In the lower Thames River, for example, where breakup is usually premature (and thence Σq is not an important variable), H_m has been found to increase with h_I, in accordance with expectation

[Beltaos, S., 1983, Ice freeze up and breakup in the lower Thames River: 1980-1981 observations, National Water Research Institute, Unpublished Report].

Fred Parkinson

Comment: Experience following similar procedures on the Liard and Mackenzie Rivers showed that H_m continued to increase with discharge - no levelling off. Ice thickness did not appear to be a key parameter; the highest jam levels occurred with the thinnest covers. Low heat input was certainly a major factor in these high jam levels.

Reply

Strictly speaking, there is no levelling off in Fig. 5 because the upper envelope of the data points increases continuously with H_m. However, the trend does suggest that levelling off might be reached at some (large) value of Q(= discharge). Similar trends have been found by the writer at other sites (eg. see Beltaos 1983). Mr. Parkinson's results suggest that large enough Q's have not been experienced in the Liard-Mackenzie Rivers for this effect to be manifested. With regard to the effect of ice thickness, it is difficult to draw conclusions unless the effects of H_F, I_Q and Q are considered simultaneously (see also my reply to Mr. Foulds and Dr. Gerard's comment).

Robert Gerard

As a follow-up to Fred Parkinson's comment on the effect of ice thickness on breakup stage on the Mackenzie: it is likely that in milder regions there is a large relative range of ice thickness and it therefore has a significant effect, whereas on the Mackenzie the relative ice thickness range is probably much less (say 20-30%), a variation that would be lost in the noise of the empirical relations being discussed.

Reply

I agree with Dr. Gerard in that large northern streams should be subject to little variation in ice thickness. In the Nashwaak River example, a three-fold variation of h_i was found. If the variation of h_i in the Liard-Mackenzie Rivers is only 20-30%, then it might be difficult to delineate

the effect of h_i on H_m (see also my replies to Mr. Foulds and Mr. Parkinson).

Henry Santeford

Comment - need to have ice thickness data at the gauge and not at a measuring site (used for discharge measurements) which is often removed from the gauge site.

Reply

This could be a matter of concern at locations where the gauge site and the discharge metering site have very different flow characteristics whereby the respective ice thicknesses might differ significantly. Moreover, if h_i is measured consistently a short distance off the gauge site, it would still provide a meaningful index. Ideally, of course, one would like to measure average h_i throughout a reach, say ten river widths long, that is centered at the gauge site.

Darryl Calkins

Did the Durham Bridge cross section have any frazil accumulation and how was it handled?

Reply

To my knowledge, there are no significant frazil deposits in the Nashwaak River at Durham Bridge.

D. Andres

Can you differentiate the effect the heat input has on both the quality (strength) of the cover and the amount and timing of runoff, which can also determine the time of breakup?

Reply

It would be very difficult to determine the effect of Σq on runoff for the present study because rainfall is often an important runoff component. In my analysis, runoff is reflected in H_B; Σq has been introduced as an

index of strength and thickness reductions that are experienced by the ice cover during the pre-breakup period (i.e. onset of mild weather to onset of breakup).

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COUPS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU-BRUNSWICK

FORECASTING THE INITIATION OF ICE BREAKUP ON THE NASHWAAK RIVER N.B.

P.W. Tang[1] and K. S Davar[2]

ABSTRACT

An operational forecasting model based on semi-empirical approaches was developed to predict the onset of ice cover breakup and the resulting maximum river stages. The empirical relationships involve a heat transfer index, ice cover thickness, river stages and flow conditions in the channel. The computational steps of the model are outlined in detail. Model validation indicated that the model performs satisfactorily for winter events accompanied by sudden traumatic river breakup. The method used in estimating the gauge height of the ice covered rivers based on discharges is presented. The operational forecast procedures and the application of the model to predict 1982-83 events are described in detail. The results demonstrated that the model could be considered as an useful operational tool.

[1]Flood Forecast Engineer, River Forecast Centre, Water Resources Branch, New Brunswick Department of the Environment, Fredericton, New Brunswick

[2]Professor of Civil Engineering Department, University of New Brunswick, Fredericton, New Brunswick

RÉSUMÉ

Un modèle fonctionnel pour prédire le début de la débâcle ainsi que le niveau d'eau maximum qui en résulte fut développé en utilisant une approche semi-empirique. Les rapports empériques comprennent un indice de transfert de chaleur, l'epaisseur de la glace, les niveaux d'eau et les conditions, d'écoulement de la rivière. Les étapes de calculs du modèle y seront indiquées en détail. L'évaluation du modèle a démontrés que celui-ci donne des résultats satisfaisants lors de crues accompagnées d'une débâcle soudaine. La méthodologie utilisée pour prédire les niveaux d'eau des rivières prises par la glace, en utilisant les valeurs calculées du débit, vous sera présentée. La façon de procéder ainsi que la mise en application du modèle seront démontrées en utilisant les données de la crue de 1982-83. Il en resulte que le modèle s'avère utiles comme outil de travail.

INTRODUCTION

An operational forecasting procedure for predicting the onset and severity of river ice breakup is valuable for river management, especially for protection from flood stages in the winter and spring time. It was the purpose of this study to develop and test such an operational model for the Nashwaak River in New Brunswick.

During summer 1980, a joint study was initiated by National Water Research Institute (NWRI) at the request of the N.B. Sub-committee on River Ice. The chief objective of that study was to determine whether the existing hydrometric gauge records could be utilized to forecast the initiation and severity of river ice breakup.

As a consequence, the hydrometric gauge located on the Nashwaak River at Durham Bridge in New Brunswick was selected for examination of stage records and analysis. The results of a preliminary study supported the use of water stage as an index to forecast breakup initiation and severity (Beltaos and Lane, 1982). This initial success prompted the development of a computer model which will be employed by the River Forecast Centre staff as an operational tool. The application of this model to 1982-83 winter spring events is presented and discussed in this paper.

DEVELOPMENT OF THE ICE BREAKUP MODEL

The development of the ice breakup model for the Nashwaak River near Durham Bridge is primarily based on the study by Beltaos and Lane (1982). The hydrometric site, data description, empirical relationships, the computer model and the model validation are described below.

The Hydrometric Site

The Nashwaak River is a major tributary of the lower Saint John River. The Nashwaak River Basin has an area of 1,760 sq. km. and lies in central New Brunswick, north of Fredericton. The topography is rugged and the entire river basin is forested, except for some agricultural land near Stanley and along the lower 30 km of the Nashwaak River. Figure 1 shows the general location of the Nashwaak River Basin.

The hydrometric station at Durham Bridge is approximately 15 km from the confluence with the Saint John River; it is located near the city of Fredericton as shown in Figure 2. The station is operated by Water Survey of Canada (WSC) and is identified by the following name and number: Nashwaak River at Durham

Bridge: 01AL002. The cross-section at the gauging site is shown in Figure 3.

Description of Data

The main source of data has been the WSC recording charts of river stage at the hydrometric station for the period 1965-1981. Other information consisted of daily discharge data extracted from WSC publications (Surface Water Data Atlantic Provinces), the meteorological data from Atmospheric Environment Service (AES) publication (Monthly Meteorological Summary for Fredericton), and the ice thickness data for part of the period from New Brunswick Department of Environment (NBDOE), NEED report 1983 and WSC (unpublished record).

Extraction of Stage Data

The data that are extracted from the hydrometric stage records are listed below:

1. the maximum stable freeze-up stage (H_F);
2. the stage at initiation of breakup (H_B);
3. the maximum stage reached during the breakup period (H_m); and
4. the "winter" peaks which constituted the lower limit for the stage required to initiate breakup.

Each breakup event consisted of H_F, H_B and H_m. The summary of the data extracted from the hydrometric gauge records is given in Table 1 (Beltaos and Lane 1982).

Calculation of the Heat Index

The daily amount of heat input (q) per unit outer area of the ice cover is used to calculate the heat index Σq and it is defined as below (Shulyakovsky, 1963):

$$q = Q_{sr} + I_{eff} + (LE + P), \qquad (1)$$

In which:

q	= total heat exchange	($J/cm^2.d$)
Q_{sr}	= short wave radiation	($J/cm^2.d$)
I_{eff}	= long wave radiation	($J/cm^2.d$)
$(LE + P)$	= heat exchange by conduction and convection plus evaporation or condensation.	($J/cm^2.d$)

Each component was further simplified as shown below (Beltaos and Lane 1982):

$$Q_{sr} = (0.175 + 0.0127\ S_u/K_d)Q' \quad (2)$$

$$I_{eff} = 35.6\ \theta\ K_d - 20.8\ S_u - 166\ K_d; \text{ and} \quad (3)$$

$$(LE + P) = K_d((39.7+21.7W)\theta - 34.8 - 63.56); \quad (4)$$

In which:

S_u = number of hours of sunshine (h),
K_d = length of day time expressed as a fraction of the 24 hour period,
Q' = maximum possible solar radiation ($J/cm^2.d$)
θ = mean daily temperature (°C),
W = average wind velocity (m/s).

The maximum possible solar radiation Q' was determined from Table 30 given by Shulyakovsky (1963) and the length of day time K_d was determined from the Observer's Handbook (1983). The monthly values of both Q' and K_d are given in Table 2.

The meteorological data S_u, W, and θ were taken from the Monthly Meteorological Summary for Fredericton prepared by AES.

With all the values determined each component Q_{sr}, Ieff and (LE + P) can then be evaluated.

The computation of the heat index Σq was started on the first day for which q was positive for a selected time period during the winter/early spring regime; it ended at the time of breakup initiation. The summation was carried out algebraically during this period, so long as Σq remained positive. If Σq became negative at any time, it was set equal to zero and the summation was initiated again on the first day of positive q.

Estimation of Ice Thickness

The ice thickness data for the Nashwaak River near Durham Bridge was obtained from NBDOE and WSC. There were seven measurements of ice thickness taken between 1965 and 1981 by WSC and they are shown in Table 1 (Beltaos and Lane 1982). NBDOE started the "Ice Thickness Measurement Program" in 1979 and the results have been published in the NEED Report (1983). Relevant information is summarized in Table 3.

In order to determine the ice thickness at the start of the mild weather spell that would lead to breakup, h_i was plotted versus the accumulated degree-days of frost, and the curve was extended during periods of missing data. The relationship is shown plotted in Figure 4.

Empirical Relationships

The rise in stage ($H_B - H_F$) required to initiate breakup of the ice cover was plotted versus the ice thickness h_I which was taken at the start of the mild weather. The upper envelope of the data points is described by the straight line ($H_B - H_F$) = 2.5 h_I as shown in Figure 5. The deviation of any one data point from the upper envelope (i.e. 2.5 h_I - ($H_B - H_F$)) was computed and plotted versus Σq as shown in Figure 6. This plot defined the empirical relationship of the stage at initiation of breakup with the heat index.

The maximum stage at breakup H_m was first plotted versus H_F. An upper envelope for values of H_m was then defined by the straight line H_m = 1.22 + 1.18 H_F as shown in Figure 7. The deviation of the value of H_m from the corresponding upper evenlope value (i.e. H_m - 1.22 - 1.18 H_F) was computed and plotted versus Σq as shown in Figure 8.

The maximum stage H_m was also plotted versus the prevailing discharge. The upper envelope for values of H_m was defined as shown in Figure 9.

These plots defined the empirical relationships for the maximum stage at breakup.

If the H_m obtained from Figure 9 was lower, it indicated that the prevailing discharge was not high enough to support the H_m as obtained from Figure 8. In the other case, if the H_m obtained from Figure 8 was lower, it indicated that the heat index was high enough to have caused the ice to reduce its maximum backwater effect. Therefore the H_m obtained as the lesser of the two values, was considered to be the most probable flood stage for the given conditions.

Computer Algorithm

After the empirical relationships were established, the procedures for estimating the stage at initiation of breakup and the maximum stage at breakup were computerized by the authors to facilitate use as an operational tool. The curves as shown in Figures 6, 8 and 9 were put into the computer model in the form of a simplified linear segments for performing the necessary computation (interpolation).

The computation steps were as follows:

(1) The heat index Σq was first computed using the meteorological data of mean daily temperature (θ), average wind velocity (W) and sunshine hours (S_u) as daily variables.

(2) The value of the expression ($2.5\ h_I - (H_B - H_F)$) was obtained from Figure 6 with a known Σq.

(3) The stage at initiation of breakup H_B was then evaluated from the expression ($2.5\ h_I - (H_B - H_F)$) based on the recorded values of h_I and H_F.

(4) The probable stage at initiation of breakup (H_B) was then compared with the observed or the predicted river stage. If (H_B) was higher, river ice breakup would not have been initiated. On the other hand, if H_B was lower, that meant the observed or the predicted river stage had exceeded the threshold value, thus river ice breakup would have been initiated.

If there was no ice breakup, steps 1 to 4 were repeated. If ice breakup was initiated, the model proceeded to determine the maximum stage at breakup H_m.

(5) The value for the expression ($H_m - 1.22 - 1.18\ H_F$) was first determined from Figure 8 with the Σq obtained from step 1.

(6) The value of H_m was then evaluated from the expression ($H_m - 1.22 - 1.18\ H_F$) based on the recorded value of H_F, the same as in step 3.

(7) The value for H_m was also determined from Figure 9 with the known prevailing discharge.

The choice of H_m was taken as the lesser of the two obtained from steps 6 and 7.

Model Validation

Model validation basically checks the predictive capability of the model. Seventeen seasons of data were used to check the model and the results are summarized in Table 5. There were 16 successful predictions out of 16 winter events. For the spring event, only eight out of seventeen events successfully predicted the time of the ice breakup events. The results indicated that the model performed satisfactorily during the winter events.

This could be explained by the ice cover during the winter event having been relatively firm while the ice cover during spring events would have been in an advance stage of deterioration due to heat input. Under these conditions, the assumptions of the model do not obtain and the river data also tend to be relatively difficult to define properly.

Since the need for predicting the maximum flood stages is greatest during periods of sudden traumatic river breakup and the model performs adequately for this type of breakup it may be considered an useful operational tool subject to future improvement.

OPERATIONAL FORECAST PROCEDURES

In making operational forecasts of river ice breakup, it is required to predict the probable gauge height a few days in advance. The method used in estimating the gauge height, the data required to make the forecast, the forecast procedures and the 1983 events are described in the following sections.

Estimation of Gauge Height with Ice Cover

Forecasts of river flows for the Nashwaak River at Durham Bridge for five days are normally available by running the SSARR (Streamflow Synthesis and Reservoir Regulation) model for the Saint John River of which it is a tributary. Since there are no rating curves for rivers with ice cover, it is necessary to use some other method of estimating the gauge height (piezometric elevation) based on the discharge.

This method is based on the assumption that under a stable ice cover condition, the rate of change of the logarithm of the "apparent flow" is approximately equal to the rate of change of the logarithm of the "effective flow".

The apparent flow (Q_a) is the "equivalent open-water discharge", it is obtained from the observed stage under ice conditions by referring to the open-water stage-discharge rating curve. The effective flow (Q_e) is the actual discharge under the ice cover.

The procedure for estimating the stage H_n at time step n from the forecast discharge is as follows:

(1) The initial apparent flow $(Q_a)_o$ is obtained from the observed stage H_o at the same time step by using the open-water rating table.

(2) The apparent flow $(Q_a)_n$ for the next uniform time step is obtained by applying the following equation:

$$(Q_a)_n = \log^{-1}(\log(Q_a)_{n-1} + \log(Q_e)_n - \log(Q_e)_{n-1}) \qquad (5)$$

where

the subscript n specifies the time step 1...n,
$(Q_e)_{n-1}$ is the effective flow, corresponding to H_{n-1}, and
$(Q_e)_n$ is the effective flow, based on the flow forecast.

(3) From the calculated apparent flow $(Q_a)_n$, the stage, H_n, is obtained by applying the same open-water rating table.

The assumption underlying equation (5) is a mathematical statement of the graphical procedure adopted by local personnel of Water Survey of Canada.

The assumption seems to hold as long as the ice cover remains undisturbed. However, it does not hold if resistance is developed under the ice cover due to large slush deposits or hanging dams since the apparent stage would increase while the flow decreased.

Data Requirement

Both, observed and forecast meteorological and hydrometric data are required in making operational forecasts of river ice breakup.

The data required to run the computer model are listed below:

(1) the stable freeze-up stage H_F (m);
(2) the ice thickness H_I at the start of the warm spell (m);
(3) observed and forecast data:
    a) mean daily air temperature (°C),
    b) sunshine hours (h),
    c) average daily wind speed (km/h),
    d) effective streamflow (m^3/s);
(4) observed river stage (m).

A corresponding set of stable freeze-up stage and ice thickness is required for each breakup event. The rest of the data are required each day after the warm weather starts until the breakup event is over.

Forecast Procedure

The corresponding stable freeze-up stage H_F and the ice thickness h_I were determined to initiate the breakup prediction. The stable freeze-up stage H_F is normally determined after a stable ice cover is formed. Periodic field measurements of ice thickness and streamflow are required to properly define the initial conditions.

Once the warm weather trend is predicted, the ice breakup model is initiated with all the necessary data. The model is updated each day with the actual observed data until the event is over.

If ice breakup does not occur, the procedures are repeated again with the same H_F and an updated h_I at a later time. If ice breakup does occur and the ice cover refreezes again (as in mid-winter events) the procedure is repeated with a new H_F value for the future event. However, if the river does not freeze-up again (as in spring events), the prediction process is terminated. The flow chart for the operational procedure is shown in Figure 10.

Prediction of Breakup Events

There were two ice breakup events which occurred during the 1982-83 season. In mid-January 1983, a warm spell led to the breakup of the ice cover and the other breakup event occurred in late March.

The formation of ice cover started to affect the gauge stage on December 13, 1982. A value of 2.50 m was chosen for the stable freeze-up stage H_F which occurred on December 19, 1982.

On January 11, 1983, a warm weather spell was predicted to occur and the Ice Breakup Model was initiated. A value of 0.21 m was estimated for the ice thickness. The observed meteorological data were obtained from the Fredericton Weather Office while the forecast meteorological data for the next five days were obtained from the weather office of the River Forecast Centre (RFC). The effective streamflow was obtained from Water Survey of Canada (WSC). It was based on the previous measured discharge. The forecast streamflows for the next five days were obtained by running the SSARR model. The observed stage was obtained by phoning the telemark at Durham Bridge.

The result of the ice breakup prediction indicated that the ice breakup would occur on January 12, with a breakup stage H_B = 2.64m and a probable maximum stage H_m = 4.0m.

Field observations showed that ice breakup occurred at 0900 hours on January 12 with H_B = 2.65m and H_m = 3.83m at 1500 hours on the same day.

After the breakup, the river froze again and a new H_F of 2.55m occurred on January 16.

A warm weather trend was predicted on March 11, 1983 and the ice breakup model was initiated again. The ice thickness was estimated to be 0.60m based on previous measurement. The

model was updated each day with actual data.

The result of the ice breakup prediction indicated that the ice breakup would occur on March 21 with H_B = 2.43m and H_m = 3.7m.

Field observations indicated that the ice breakup commenced at 1000 hours on March 22 with H_B = 3.30m; maximum stage at breakup H_m = 3.98m occurred at 1800 hours.

DISCUSSION

The results of the 1983 predictions indicated that the model has potential for use as an operational tool. It is a simple model and the operational procedures are straight forward and easy to follow.

These results as summarised in Table 6 indicated that the model performed satisfactorily for the winter breakup event (January 12, 1983). This is in agreement with the model validation, carried out for previous historical events during the period 1966 - 1982.

Also it should be realized as evident from the model validation runs that the model did not perform well when the ice cover deteriorated gradually insitu. This is because under such conditions, the stage at initiation of breakup could not be defined properly.

It is important to point out that the accuracy of the information extracted from the hydrometric stage record is the key to the success of this model.

In the forecast environment, the success of the prediction depends on the accuracy of the weather forecast, the flow forecast and the corresponding estimate of the gauge height with ice cover.

CONCLUSION

An acceptable ice breakup model has been developed for the Nashwaak River near Durham Bridge based on the Shylyakovsky-Beltaos methodology.

The performance of the model is encouraging during winter events resulting in sudden traumatic river breakups.

The 1982-83 events demonstrated that the model can be considered as an useful operational tool with potential for improvement.

RECOMMENDATIONS

The time interval used for the model was one day. The model could be improved by using shorter time intervals and analyzing the incremental increase in accuracy. This will enable the forecaster to forecast more accurately the time of the day the breakup would most probably occur.

The empirical relationships plotted in Figures 8 and 9 should be simplified by analyzing H_m versus Q with Σq as a third variable.

REFERENCES

Ashton, G.D. 1973, "Heat Transfer to River Ice Covers", Proc. Eastern Snow Conference, pp. 125-35.

Beltaos, S., 1983. "Guidelines for Analysis of Hydrometric Gauge Records", National Water Research Institute, Unpublished Report.

Beltaos, S. and Lane, RI, 1982. "Ice Breakup characteristics of the Nashwaak River at Durham Bridge, N.B.", National Water Research Institute, Unpublished Report.

Bishop, R.L., 1983 "Observer's Handbook", The Royal Astronomical Society of Canada.

Gerard, R., 1979, "River Ice in Hydrotechnical Engineering. A Review of Selected Topics", Proc., Can-Hydrology Symposium 79, Vancouver, NRCC No. 17834.

LeBrun-Salonen, M.L. 1983. "Average Solid Ice Thickness in New Brunswick Rivers". A report prepared under the New Employment Expansion Development Program (NEED).

Michel, B., 1971. "Winter Regime of Rivers and Lakes". U.S. Army Corps of Engineers, Cold Regions Monograph III-Bia.

Rosenberg, H.B. and Pentland, R.L., 1983. "Accuracy of Winter Streamflow Records". I.W.D.

Shylyakovsky, L.G., 1963. "Manual of Forecasting Ice Formation for River and Inland Lakes". Translated from Russian, Israel Program for Scientific Translations, 1966.

Tsang, G., 1982. "Frazil and Anchor Ice-A Monograph". NRCC Associate Committee on Hydrology, Subcommittee on Hydraulics of Ice Covered Rivers, Ottawa.

TABLE 1 SUMMARY OF BREAKUP CHARACTERISTICS AND ASSOCIATED PARAMETERS: NASHWAAK R. AT DURHAM BRIDGE 1965-81 (BELTAOS & LANE 1982)

Season	H_y (m)	Time of H_y (day/month)	Winter Peaks: H (m)	Time of H	Σq (J/cm^2)	Breakup: Range of H_b (m)	Probable H_b (feet)	Time of H_b	Σq (J/cm^2)	Σq Calculated (day/mo.)	Ice Thickness: h_i (m)	Time of Measurement	Maximum Stage: H_m (m)	Time of H_m	Discharge (m^3/s)	Remarks
1964-65	2.96	13/1	2.30	7/3	2263											Breakup undefinable
1965-66	1.60	29/12					1.58	22/3	2807	22/3			2.23	26/3	63	
1966-67	1.80	27/12					1.71	7/4	5992	7/4	0.70	16/3	1.73	11/4	15	
1967-68	2.34	1/1					3.25	4/2	461	4/2	0.37	12/1	3.87	4/2	100	h_i not representative of breakup value
1967-68	3.35	4/2	2.78	25/3	3855											
1968-69	2.25	29/12	1.64	27/1	0		2.71	30/3	6411	30/3			3.48	1/4	93	
							1.87	28/3	3562	28/3			1.87	30/3	22	Difficult to define breakup
1969-70	3.44	1/1				1.98 - 5.31		2-4/2	587	3/2			5.31	4/2	697	- " -
1969-70	2.29	16/2				1.43 - 1.80		27/3	4819	27/3			2.05	28/3	34	- " -
1970-71	1.65	10/12	2.07	15/3	251	0.90 - 1.31		3-4/4	6872	3/4	0.64	11/3	1.96	14/4	49	
			1.83	18/3	3179				7562	4/4						
1971-72	1.19	7/12	1.34	13/12	251	1.62 - 2.19		17-19/3	1048	17/3	0.72	6/3	2.49	20/3	28	
			1.43	16/1	0				1467	18/3						
			1.59	16/2	1383											
			1.74	12/3	0											
1972-73	2.72	14/12	2.79	5/2	0	1.91 - 2.35		17-18/3	5363	18/3			2.61	19/3	76	
			2.16	13/3	3352											
1973-74	2.20	10/1	2.22	8/3	3017	1.56 - 2.10		4-7/4	2682	4/4	0.70	27/3	2.19	7/4	56	
									4735	7/4						
1974-75	2.19	4/1	2.15	22/3	2137	1.48 - 1.68		4-6/4	4482	4/4			1.68	8/4	32	Difficult to define breakup
									6834	8/4						
1975-76	2.46	14/12	3.38	29/1	277		2.19	27/3	4864	26627/3(avg)	0.72	2/3	2.94	29/3	126	
1976-77	1.89	12/12	2.69	27/12	0	1.40 - 1.53		27-28/3	6411	27/3			2.29	31/3	74	Difficult to define H_y and H_b; H_y= 2.29 on 29/12 might be better
			1.91	16/3	3645											
1977-78	1.82	11/12				2.15 - 2.51		26/12	21	26/12			2.51	26/12	42	Probably thin ice
1977-78	2.74	18/1				1.42 - 1.54		11/4	7123	11/4			1.76	15/4	47	Difficult to define H_y
1978-79	1.18	27/11				< 6.53		3/1	293	2/1			2.57	3/1	74	
1978-79	3.48	7/1	3.26	31/1	1889	1.54 - 2.85		5-8/3	922	5/3			3.30	26/3	207	Difficult to define breakup
									2179	7/3						
1979-80	2.35	4/1				2.71 - 3.01		18-19/3	629	18/3	0.55	13/3	3.03	19/3	105	
1980-81	2.41	17/12				1.81 - 2.11		19-23/3	712	18/3			2.12	23/3	68	
									2556	23/3						

TABLE 2 MONTHLY VALUES OF MAXIMUM POSSIBLE SOLAR RADISTION Q' AND LENGTH OF DAY TIME K_d

Month	Q' ($J/cm^2.d$)	K_d (fraction)
Sept.	2035	0.525
Oct.	1440	0.458
Nov.	971	0.394
Dec.	720	0.361
Jan.	780	0.375
Feb.	1256	0.428
Mar.	1867	0.494
Apr.	2512	0.563

TABLE 3 AVERAGE ICE THICKNESS AND ACCUMULATED DEGREE DAYS OF FROST

DATE	h_i (m)	$\Sigma\theta$ (°C.d)
1979 JAN. 3	.30	289
JAN.17	.27	433
FEB. 8	.40	554
FEB.20	.61	760
FEB.28	.64	791
MAR.12	.58	778
MAR.23	.54	798
1979 DEC.22	.15	126
1980 JAN.17	.27	247
JAN.31	.37	350
FEB.14	.49	484
FEB.28	.54	594
MAR.13	.55	695
1981 JAN. 8	.24	411
JAN.14	.37	502
JAN.20	.46	587
FEB. 5	.46	712
FEB.19	.49	780
1982 JAN.15	.40	274
JAN.29	.43	502
FEB.12	.55	627
MAR.12	.64	870
MAR.26	.58	880
1983 JAN.28	.24	329
FEB.11	.35	414
FEB.18	.38	485
FEB.25	.42	545
MAR.18	.40	570

TABLE 4 DATA USED IN THE EMPIRICAL RELATIONSHIPS FOR THE NASHWAAK RIVER AT DURHAM BRIDGE

SEASON	H_r (m)	H_s (m)	H_m (m)	h_r (m)	Σq (J/cm^2)	Q (m^3/s)
1965-66	1.40	1.58	2.23	0.70	6516	83
1965-67	1.80	1.71	1.73	0.69	5403	15
1965-68	2.34	3.25	3.87	0.45	570	100
	3.35	2.74	3.40	0.50	6836	93
1968-69	2.25	1.87	1.87	0.60	4865	22
1969-70	3.44	1.98-5.31	5.31	0.60	1092	697
	2.29	1.43-1.80	2.05	0.40	4190	34
1970-71	1.65	>2.07	>2.07	0.60	384	28
	1.65	0.90-1.31	1.96	0.64	7166-7971	49
1971-72	1.19	>1.43	>1.43	0.43	0	11
	1.19	>1.59	>1.59	0.55	1427	11
	1.19	>1.74	>1.74	0.73	0	11
	1.19	1.62-2.19	2.49	0.64	1063-1658	28
1972-73	2.72	>2.79	>2.79	0.50	56- 426	88
	2.72	1.91-2.35	2.61	0.58	5037-5614	76
1973-74	2.20	>2.22	>2.22	0.60	2716	28
	2.20	1.56-2.19	2.19	0.70	1777-3531	56
1974-75	2.19	>2.15	>2.15	0.60	2802	42
	2.19	1.48-1.68	1.68	0.60	2445-4222	32
1975-76	2.44	>3.38	>3.38	0.50	485	42
	2.44	2.19	2.94	0.60	4420-5402	126
1976-77	2.29	1.49-1.53	2.29	0.55	7194	74
1977-78	1.82	2.15-2.51	2.51	0.18	47	42
	2.74	>3.33	>3.33	0.30	264	107
	2.74	1.42-1.54	1.76	0.40	7176	47
1978-79	1.10	<1.93	2.57	0.30	590	74
	3.40	>3.26	>3.26	0.35	1337	75
	3.40	>3.12	>3.12	0.64	3023	116
	3.40	2.07-2.56	3.20	0.64	5534-7385	207
1979-80	2.25	2.71-3.01	3.03	0.55	579	105
1980-81	2.41	>2.03	>2.03	0.45	548	15
	2.41	1.81-2.11	2.12	0.47	1188-2979	68
1981-82	2.18	2.50	3.00	0.64	5281-5601	85

Note:> means winter peak with no ice cover breakup; it represents a lower limit for the stage required to initiate breakup.

< means the upper limit for the stage required to initiate breakup.

TABLE 5 RESULTS OF MODEL VALIDATION

PREDICTION OF BREAKUP OCCURRENCE	No. OF VALIDATION EVENTS WINTER	SPRING	TOTAL	(%)
Successful (± 24 hours)	16	8	24	73
Less Successful (± 48 hours)	0	3	3	9
Not Successful (> 48 hours)	0	6	6	18
	16 +	17 =	33	100

TABLE 6 SUMMARY OF PREDICTION RESULTS FOR THE 1983 BREAKUP EVENTS

PERIOD	PARAMETERS	OBSERVED VALUES	PREDICTED VALUES
Winter	H_B(m)	2.65	2.64
	Occurrence of Breakup	12 January	12 January
	H_m(m)	3.85	4.00
Spring	H_B(m)	3.30	2.43
	Occurrence of Breakup	22 March	21 March
	H_m(m)	3.98	3.70

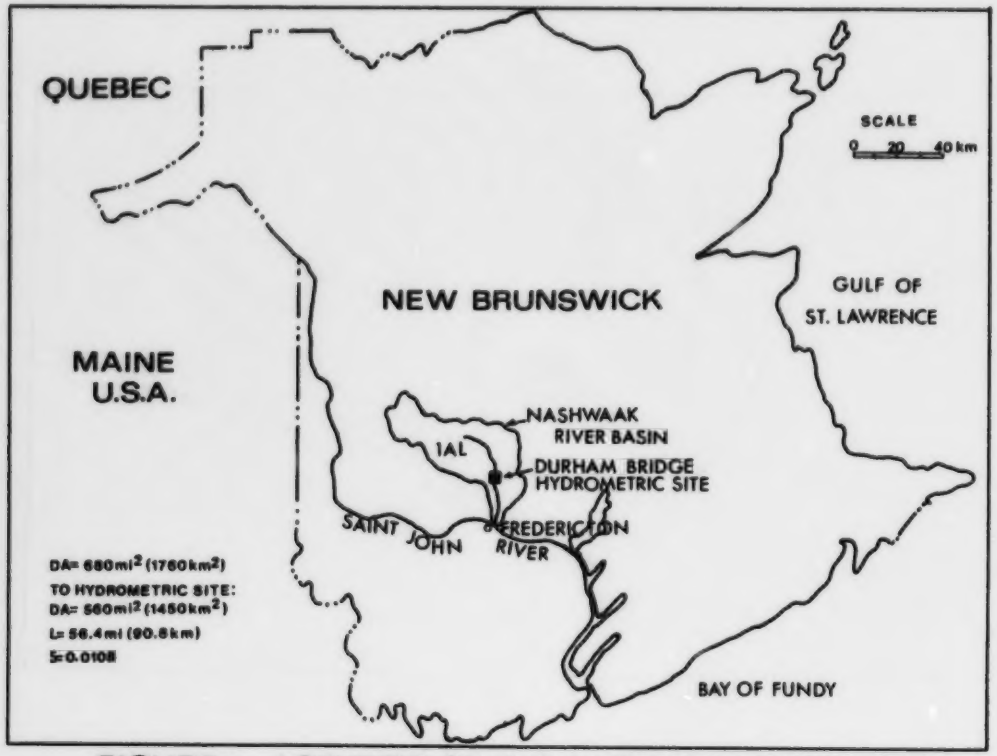


FIGURE 1 LOCATION OF THE NASHWAAK RIVER BASIN

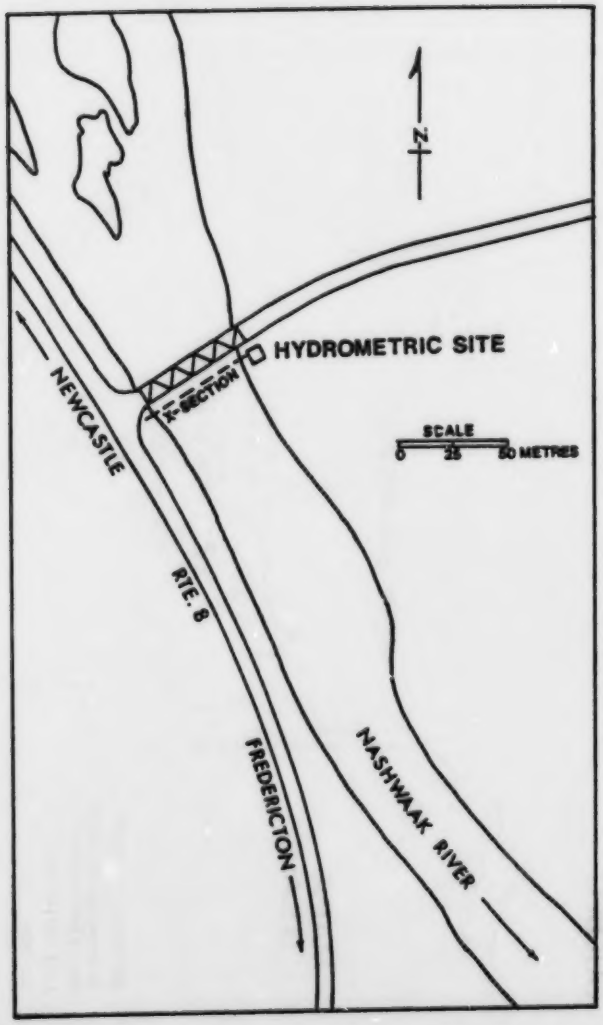


Fig. 2 Location of Cross-section of Nashwaak River at Durham Bridge

DATE OF SURVEY: 1981 APRIL 20
WATER SURFACE ELEVATION: 18.762m
DISCHARGE Q= 74.3cms
GAUGE HEIGHT= 1.569m

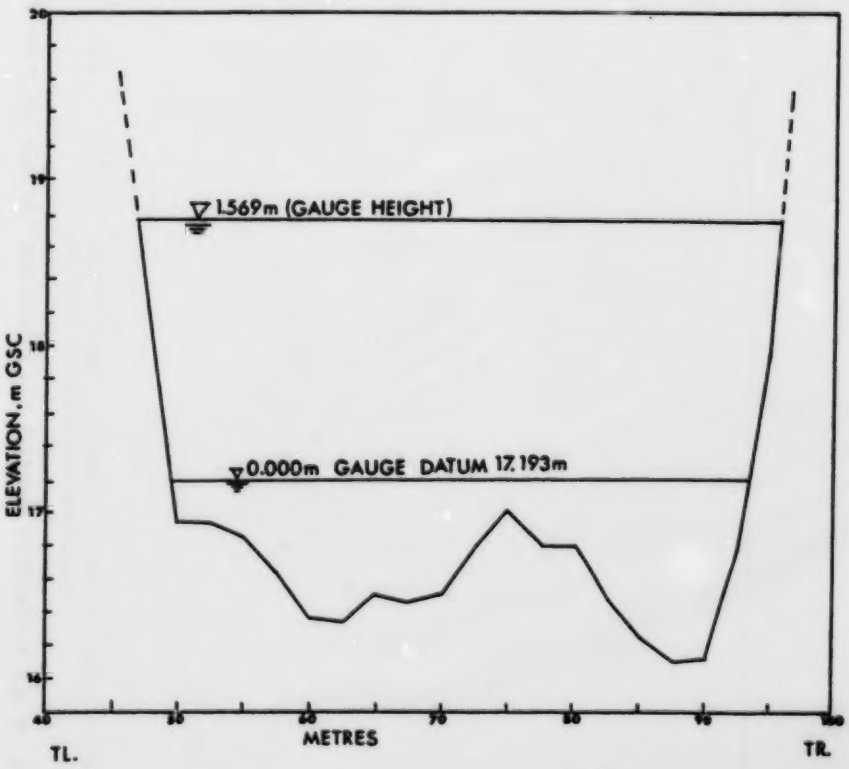


Fig. 3 Cross-section of Nashwaak River at Durham Bridge

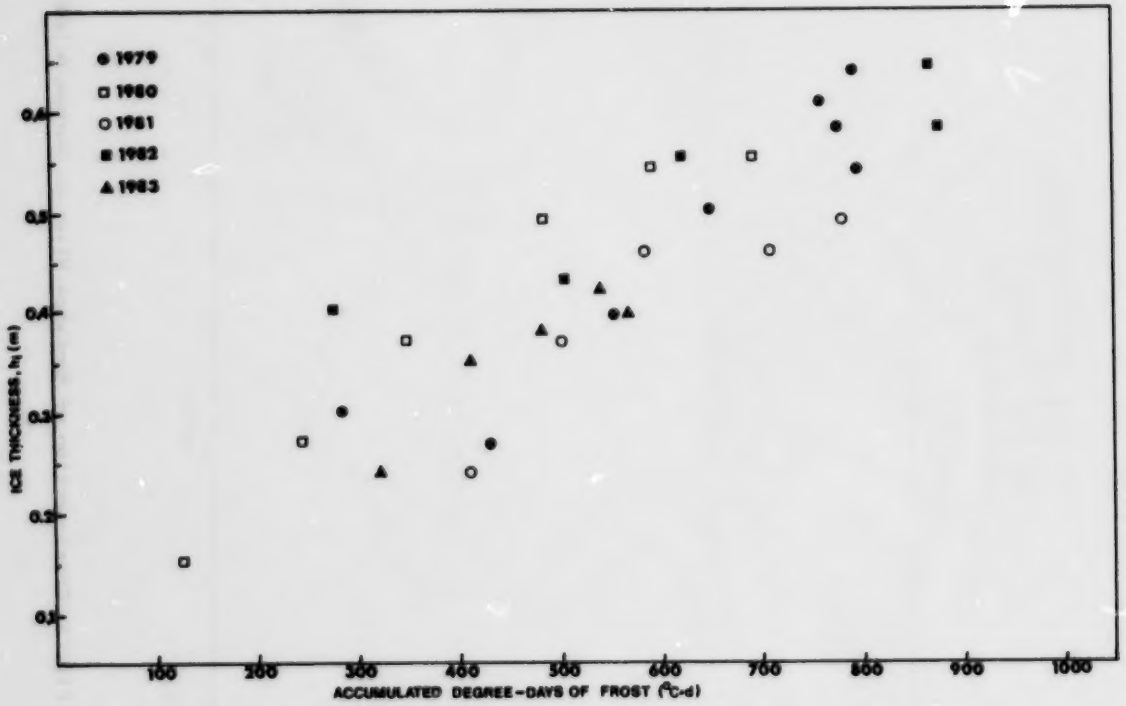


FIG. 4 ICE THICKNESS VERSUS ACCUMULATED DEGREE-DAYS OF FROST

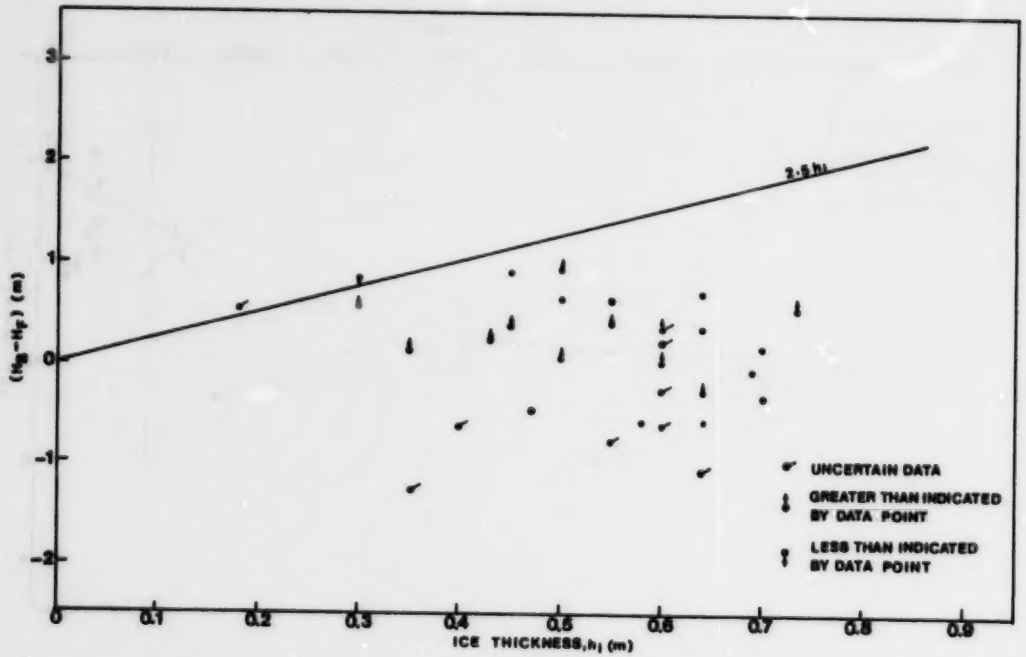


FIG. 5 VARIATION OF $(H_B - H_F)$ WITH h_I

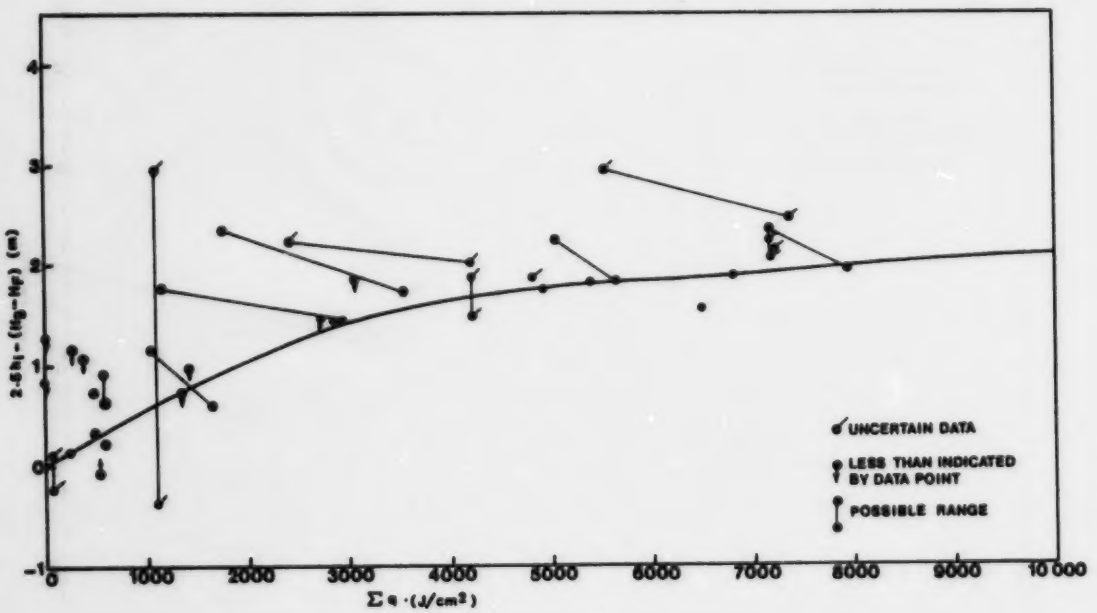


Fig. 6 Correlation of [2.5 h_f-(Hg-Hp)] with Σq

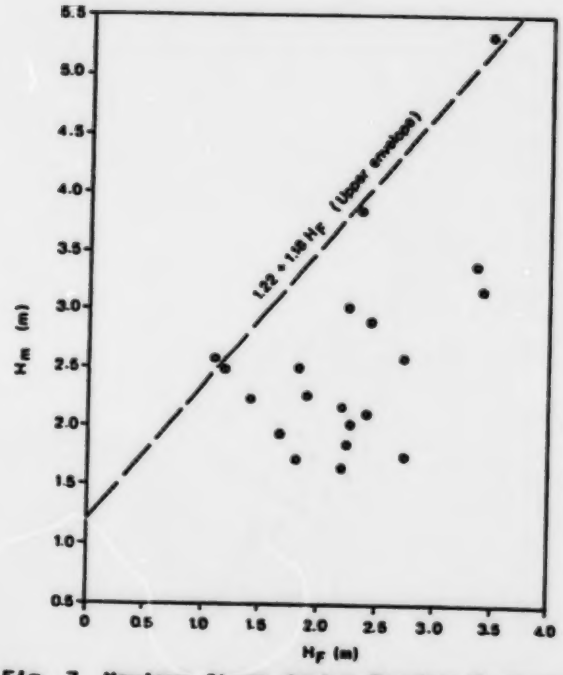


Fig. 7 Maximum Stage during Breakup H_m Versus H_F

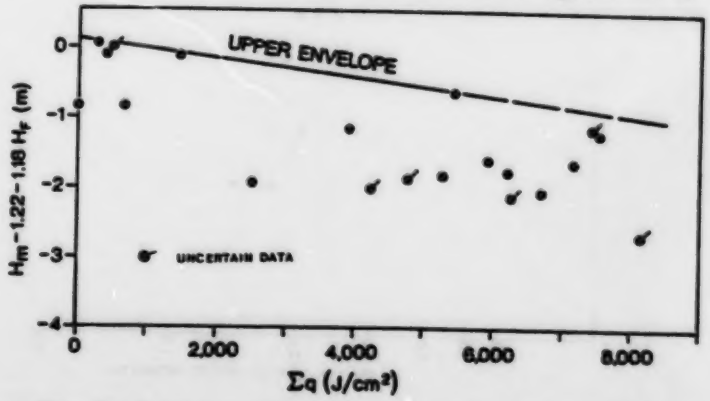


Fig. 8 Envelope Curve for (H_m-1.22-1.18H_F) Versus Σq

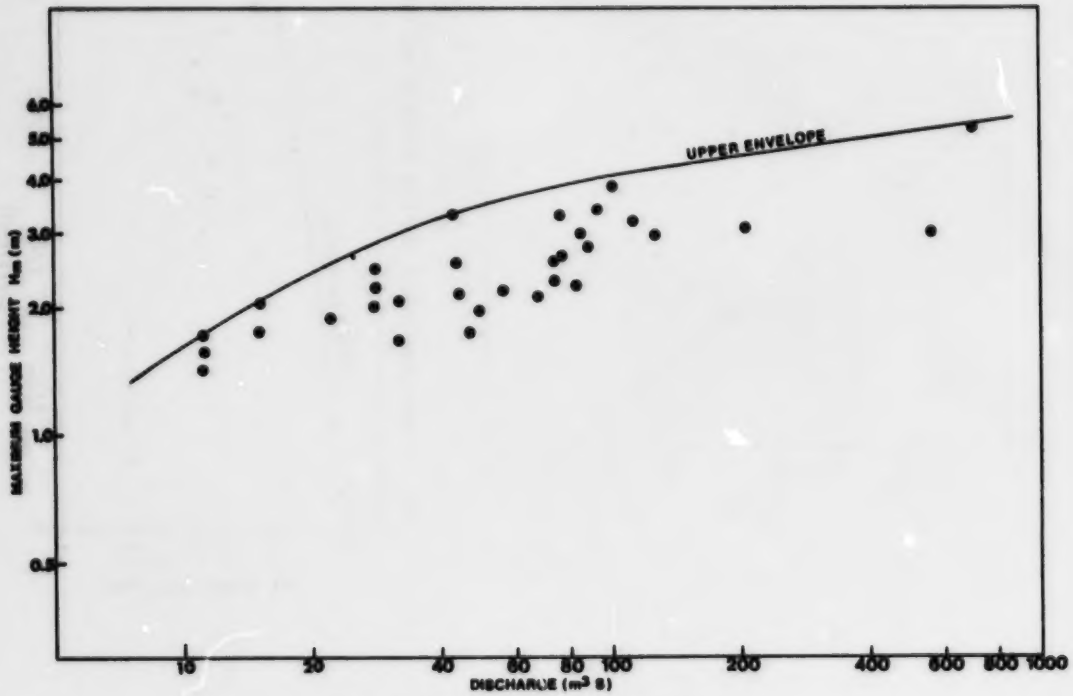


Fig. 9 Envelope Curve for H_m Versus Q

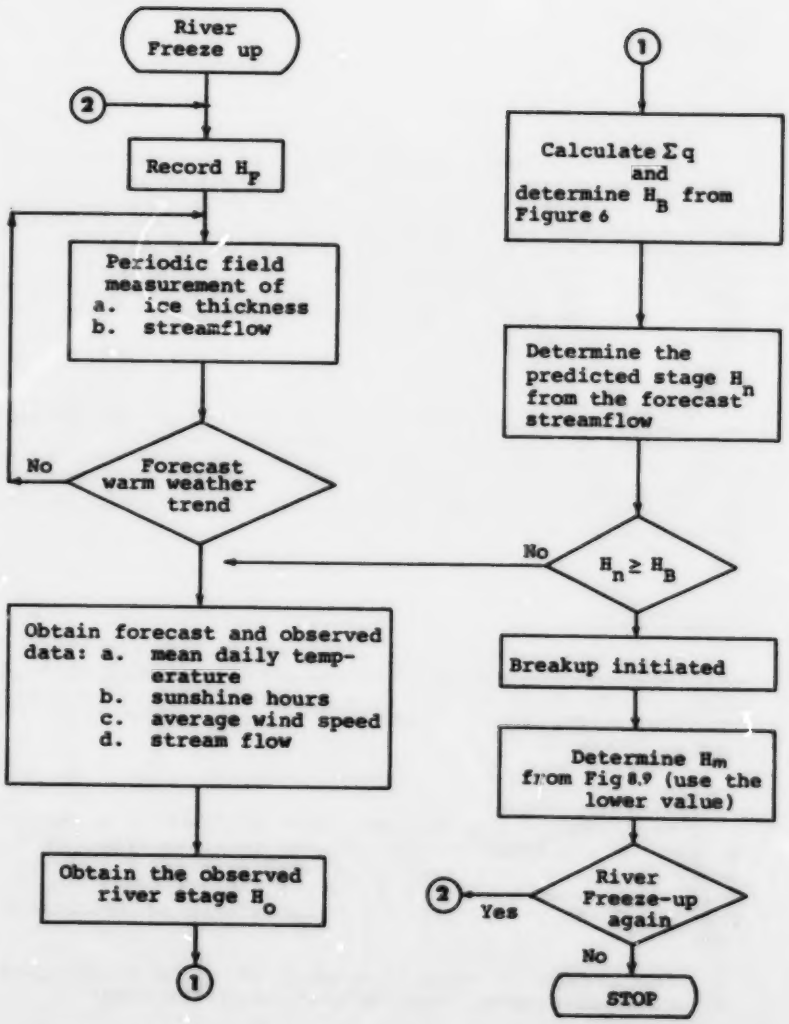


FIGURE 10 FLOW CHART FOR THE OPERATIONAL PROCEDURE

DISCUSSION

Fred Parkinson

I wish to comment concerning the proposal that thicker ice may have existed for the 1983 breakup as a result of earlier partial breakup. I would doubt that any such earlier thickening of cover with broken ice would in fact freeze thicker to affect final breakup parameters. Observations on the Liard-Mackenzie Rivers found exactly the same solid ice thickness in spring in areas that had been filled with slush ice at freeze up and other areas that started with open water.

Reply

It is likely that as a result of the earlier partial breakup of the cover, the fragmented blocks were packed together helterskelter, leading to an effective thickening of the cover on refreezing. This new thickness should be different from that developing normally from thermal-dynamic effects only.

Derek Foulds

Suggestion: The apparent difference between Arctic breakup conditions and New Brunswick ones may yet be understood with more "Arctic" data. In the "South" one can get substantial differences from year to year depending on the rate of breakup.

Reply

The rate of breakup has been studied to a certain extent in the paper "On Estimating the Likelihood of Ice Jams on the Saint John River using Meteorological Parameters". This paper was presented by Mr. P. W. Galbraith during the 5th Canadian Hydrotechnical Conference at Fredericton 1982.

The effect of rate of breakup could be investigated in further development and improvement of the model.

Robert Gerard

Further to Spyros Beltaos' remarks about refining the estimate of ice thickness from 0.6 to 1.0 m to allow for an in-place ice jam represents only a 50-60% difference. In discussion of the previous paper it was felt this would be more-or-less lost in the noise of the relations. Is it then reasonable to appeal to this difference to explain the error in predicted breakup stage? Perhaps this difference is simply characteristic of the noise too.

Reply

The physical increase in estimated ice thickness from 0.6 to 1.0 metres after partial breakup and refreezing could possibly affect the final breakup processes. Whether this effect is significant compared to the "noise" in an analytical methodology can only be determined with much more data relevant to this type of phenomenon.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21,1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

HYDRAULICS OF BREAKUP

H. S. Santeford[1] and G. R. Alger[2]

ABSTRACT

Many earlier investigations of breakup have used physical features of the ice cover, as opposed to flow conditions, as the main indicator of the breakup process. Since a number of distinctly different flow conditions can cause the same appearance of the ice cover, some confusion has evolved concerning the hydraulic characteristics necessary to initiate breakup.

Here breakup will be defined as the unsteady, non-uniform flow condition associated with the transition from fixed ice cover to semi-free surface flow. The initiation of breakup is defined as the instant in time when the flow first changes from steady to unsteady condition in an acceleration mode. Breakup is completed when the flow can again be analyzed using the steady state assumptions.

Using the above definition, a mathematical model is developed which relates the winter stage just prior to the spring runoff and the thickness of the ice to the stage, necessary to initiate breakup. The model is applied to field data obtained from several streams located in the upper peninsula of Michigan.

A discussion of other related "ice out" conditions is also provided.

[1] Civil Engineering Department, Michigan Technological University, Houghton, MI 49931.

RÉSUMÉ

Maintes investigations anciennes de la fragmentation de la glace ont employé des caractéristiques physiques de la couverture de glace au lieu des conditions d'écoulement comme indicateur principal du procédé de fragmentation. Puisque plusiers conditions bien differentes peuvent présenter le même aspect à la glace, une certaine confusion a developé en ce qui concerne les caracteristiques hydrauliques qu'il faut pour commencer la fragmentation.

Ci-après on va définir la fragmentation comme la condition d'écoulement à la fois irregulière et instable qui ont rapport à la transition entre la couverture fixe de la glace et l'écoulement semi-libre de l'eau de surface. On a defini le commencement de la fragmentation comme le premier moment quand l'écoulement change d'une condition stable à une condition instable dans un mode d'accélération. La fragmentation est terminée quand il devient encore possible d'analyser l'écoulement en supposant qu'ily ait une condition stable.

Tout en employant la définition ci-dessus, on a developé une modèle mathématique qui établit un rapport entre a) le niveau hivernal juste avant le ruissellement du printemps et l'épaisseur de la glace, et b) le niveau et ainsi le débit relatif à la fragmentation. On a appliqué cette modèle aux données obtenues de plusiers ruisseaux situés dans la peninsule supérieure de Michigan.

Aussi, on a fourni une discussion des autres conditions relatives à la fragmentation.

INTRODUCTION

To people living along northern rivers, the occurrence of breakup is often a major event. In some cases such as the Nenana Ice Classic in Alaska, breakup is a time to rejoice and make merry. In other cases breakup can be a time of flooding and disaster. Regardless of the circumstances, several key questions are almost always associated with the approach of breakup. Namely: (1) When will breakup occur? and (2) How severe will it be? These same two questions have been asked for almost as long as man has lived along the northern rivers. Yet relatively speaking, very little research has been done to address these questions. Much of the work that has been done has been of an empirical nature correlating meteorological parameters to some observed feature of the ice cover, such as the initial movement.

A recent paper by Beltaos (1983) is noteworthy in that he attempts to relate hydraulic parameters to the initiation and the severity of breakup. Beltaos bases his analysis on the maximum stage at freeze-up and considers a dimensionless grouping of the parameters. He states "the model is incomplete because it neither predicts the mathematical form of the various dimensionless functions involved, nor does it account for thermal deterioration of the ice cover during the pre-breakup period."

In terms of this presentation, the cited works of Beltaos were most significant as they formed to starting point - namely what is the mathematical form of the function defining the breakup process.

DEVELOPMENT OF THE MODEL

In attempting to develop the mathematical formulation of the breakup process it was necessary to deviate somewhat from the definitions used by Beltaos as well as many other previous authors. Here, *breakup* will be defined as the unsteady, non-uniform flow condition associated with the transition from fixed ice cover to semi-free surface flow. The *initiation of breakup* is defined as the instant in time when the flow first changes from steady to unsteady conditions in an acceleration mode. Breakup is completed when the flow can again be analyzed using the steady state assumptions. Breakup could be complete even though ice floes are still present on the channel. Furthermore, on large rivers with long distances between successive ice controls, an "ice out" condition can occur prior to breakup for the reach. The *ice control* for a reach of river is defined as the location along the reach where the combination of physical features of the ice and channel dictate the energy gradeline slope at all other points of the reach. The ice control may change from time to time throughout the winter season.

Breakup can further be categorized into two major types: thermal, and premature. With the *thermal breakup*, the thermal inputs into the ice

are sufficient to melt and weaken the ice to such an extent that the drag force exerted by the flow is enough to fracture the ice and transport it downstream with the flow. The *premature breakup* results from a rapid increase in runoff and consequently a rapid increase in flowrate. The stream response is sufficiently quick that the strength of the ice is greater than the forces exerted by the flow. With the premature breakup, the initiation is governed by the flow and is independent of the strength properties of the ice. The premature breakup is the only one which will be considered in this discussion. It represents the maximum stage and flow conditions which can exist due to an ice cover.

All breakups will consist of an initiation and acceleration phase. At times there can also be a deceleration or stalling of the flow. This deceleration phase is generally referred to as the ice jam. From a hydraulic standpoint, it is distinctly different from the acceleration phase. The hydraulics of the ice jam are very similar to the hydraulics of freeze-up and are considered beyond the scope of the discussion. The concerned reader should see Santeford and Alger (1984a, 1984c) for a discussion of the freeze-up model.

Initiation of Breakup

For the initiation of the premature breakup two conditions must be met. First, in order for the ice cover to move, there must be a place for it to move into. In the natural system, the velocity of flow just downstream of a control section often exceeds the critical velocity above which an ice cover can not form. Even where an ice cover does form downstream of a control section, the increased velocity results in a thinner ice sheet which is more responsive to thermal degradation. A similar condition exists in bends and other locations where increased currents and upswellings result in a thinner ice cover which respond to thermal degradation prior to the main breakup period. These small areas of open water and/or reaches of thinner, weaker ice play a key role in the breakup process.

The second condition that must be met is that the bottom of the ice sheet must be raised to a level at least equal to the elevation of the top of the shore fast ice which parallels each bank. Figure 1 shows a schematic diagram of the stage conditions associated with the initiation of breakup. If GH_w is the indicated winter stage which existed just prior to the increasing flow event which caused breakup and t is the ice thickness, then the critical stage for the initiation of breakup at the ice control, GH_c, is given by:

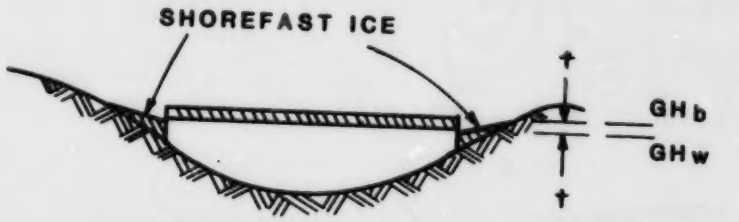
$$GH_c = GH_w + t \qquad (1)$$

If the gage is located at or near the ice control and further there is a place for the ice to move into, then conditions will be defined as a Type I breakup and, from a practical standpoint, the stage associated with the initiation of breakup, GH_B, is:

$$GH_B = GH_c = GH_w + t \qquad \text{Type I} \qquad (2)$$

During the period that the flowrate and stage are rising, the flow is governed by the ice control. The flow conditions are identical to those which existed for the period of stable ice control. Recent work by the authors (Santeford and Alger, 1984b) has shown that the under ice flowrate is directly related to indicated stage and that such relationships are fully applicable up to the time of the initial movement of the ice, i.e., the initiation of breakup. From the continuous stage records, either historically or in real time, one would know GH_w. The thickness of the ice, t, is either known from measurement or can be estimated. Thus, for this type of response and the premature breakup, the stage and flowrate necessary to initiate breakup are both predictable quantities.

WINTER CONDITIONS



INITIATION OF BREAKUP

Figure 1. Schematic diagram of conditions associated with the initiation of breakup at the ice control (Type I).

In the natural system a gage is almost always situated at a location removed from the ice control. The increased resistance and the buoyant displacement of the ice result in a water surface profile different from the open water case and governed by the ice control. If conditions remain stable, then as the flowrate and stage increase at the ice control, an equal increase is occurring at the gage location. Thus here again,

$$GH_B = GH_w + t$$

A completely different type of response can be noted at the initiation of breakup. Under some conditions an ice cover will exist on a given reach of river solely because of the overriding influence of a downstream condition. For example, a tributary may develop an ice cover because the ice cover on the main stream initiates the surface stalling of the floating ice on the tributary. The ice cover on the tributary affects flow characteristics during the period of stable ice control. However, the true ice control section is located on the main stream. Equation 2 above defines the conditions necessary to initiate breakup on the mainstream. Once the mainstream ice is free to move, breakup on the tributary can be initiated because the retarding force holding the tributary ice in place has been removed. In such cases a gage located on the tributary may give no indication of conditions leading up to the initiation of breakup. This condition will be classified as a Type II breakup.

Transition Period

Once the breakup has been initiated, i.e. the ice at the control is free to move, the resistance caused by the ice is suddenly reduced.

Upstream of the ice control the specific energy at any point is greater than that necessary to substain the flow with a moving ice cover. Consequently, the flow rapidly accelerates converting what had been potential energy associated with the increased depth to kinetic energy associated with an increased velocity. In doing so, the stage upstream of the ice control drops while that downstream rapidly increases. A negative surge is propagated upstream while a positive surge is propagated downstream. Figure 3 is a schematic diagram illustrating the wave formation associated with the ice movement.

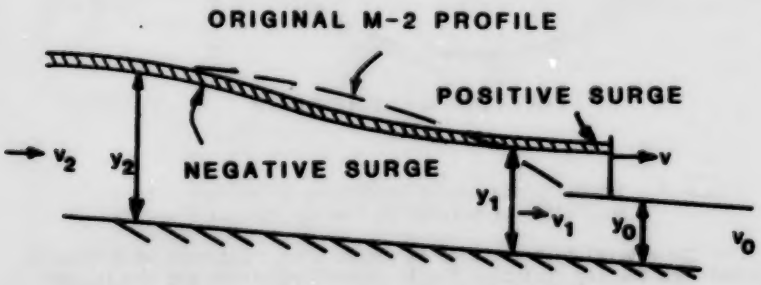


Figure 2. Wave formation associated with ice movement.

Prior to the initiation of breakup, the ice cover was a rigid body with a defined mass and inertia. It can not instantly move with the same speed as the flow. Rather, the water attempting to form the positive surge will encounter a retarding force caused by the ice. Defining the depths and velocities as shown in Figure 2; the effects of the ice by the term $F_i/B_o \gamma y_o$, where F_i is the force on the ice, B is the section width, and γ is the unit weight of water, the following equations can be written:

$$\frac{(v - v_o)^2}{y_o g} = \frac{(1/2)\left[(y_2/y_o)^2 - 1\right] \cdot F_i/b\,\gamma\, y_o^2}{1 - y_o/y_2} \quad \text{(momentum eqn. for positive surge)}$$

$$v_2 + 2\sqrt{gy_2} = v_1 + 2\sqrt{gy_1} \quad \text{(energy eqn. for negative surge)}$$

$$v\,(y_1 - y_o) = v_1 y_1 - v_o y_o \quad \text{(continuity eqn.)}$$

The above analysis is similar to that proposed by Henderson and Gerard (1981) except that here the forces exerted by the ice have been included. The effects of including the ice forces in the momentum equation causes the velocity of the positive surge, v, to be less than the free water condition, i.e. $v < \sqrt{gy}$. Also, the downstream stage associated with the positive surge, y_1, will increase beyond the halfway point between y_o and y_2 and will approach the original upstream depth, y_2. When the ice forces are included one obtains 3 equations in 4 unknowns. However, field observations should define the range of velocities of the positive surge which in turn would allow for a more precise estimate of the maximum downstream stage response.

The positive and negative surges show in Figure 2 will occur with any movement of the ice cover. With the initiation of breakup, the ice cover upstream of the release point is often very close to the shore fast ice and the difference in depths y_o and y_2 may be small. Consequently, at times the ice will initially move a short distance and lodge again. However, once the flowrate is large enough to satisfy Eqn. 2 and the rate of rise is sufficient to offset the initial stalling, the movement of the ice will be substained and a period of unsteady flow initiated.

For all locations upstream of the ice control, there must be a rapid decrease in stage associated with the passage of the negative surge. There will also be an acceleration of the mean velocity and an increase in flowrate. During this transition period of unsteady flow, the open water an/or ice covered stage-discharge relationship(s) are not valid. The dewatering of the under-ice storage is indicated by a drop in stage but an increase in flow.

In many instances the entire breakup process is associated with a short time period of decreasing stage with rapidly changing flowrates. If the flood hydrograph which initiated the breakup process is sufficiently strong the increasing stages of the flood hydrograph quickly override the decreasing stages caused by the negative surge and the stage again begins to increase. In many instances (and probably the vast majority) the breakup process is complete. The flow has returned to the normal condition associated with a conventional flood hydrograph and the open water stage-discharge relationship is again valid even though small ice floes may be present on the surface. A schematic representation of the stage hydrograph and flow hydrographs associated with this type of breakup are shown in Figure 3.

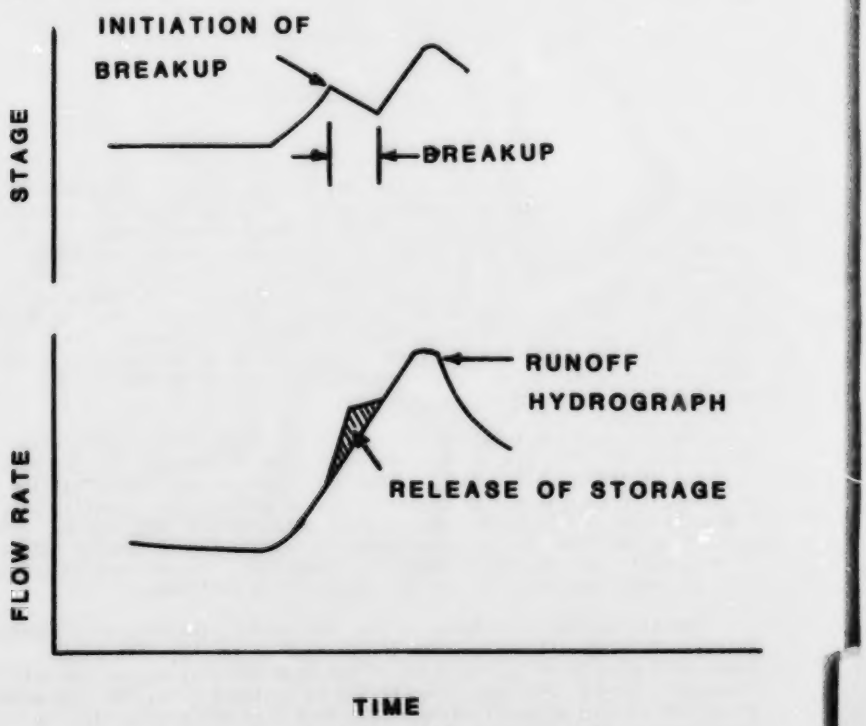


Figure 3. Schematic representation of the stage and flowrate hydrographs associated with a normal breakup.

Downstream of the ice control, the positive surge is additive to the increasing stage of the flood hydrograph. On many of the smaller streams where the individual reaches are relatively short, a downstream gage only sees a rising stage. From the stage hydrograph for a downstream gage, it is not possible to ascertain if the rise in stage is from: (a) the passage of a positive surge from an upstream release; (b) an increase in runoff; or (c) some combination of the two. Furthermore, in many instances, it really doesn't matter. If the ice in a particular reach remains relatively intact, it offers a similar resistance to the flow as it did during the pre-breakup period. Thus, the initiation of the breakup in the lower reach of river is hydraulically the same as described above.

In contrast, rivers with long distances between successive ice controls (usually large rivers) often behave differently that describe above. For example, on the Yukon and Kuskokwim in Alaska, the distance between successive ice controls is sufficiently long that a reach of the river can accommodate the entire release surges. An observer (or gage) located along the upper portions of the reach will observe the passage of the wave, often with an accompanying total fracturing of the ice cover. (Gerard reported on this condition at a recent ASCE/CSCE meeting at Edmonton - See Kent et. al. 1984). In such cases, the true premature breakup of the reach can not occur until the stage at the ice control for the reach has obtained the value of given by Eqn. (2), i.e. until the positive surge or flood wave reach the ice control. In the interim, the increasing flowrate associated with the slower moving snowmelt hydrograph causes some of the upstream ice which had been fractured by the passage of the release surge to become entrained with the flow and flushed under the still existing downstream ice cover. On the Yukon River it is common to see a 100 km or more reach of river ice become fractured and flushed out of the system while a solid ice cover still exists at the ice control for the reach. An observer looking only at a particular section of the river, might conclude that since the ice is gone, breakup had occurred. However, from the above definitions, breakup on a reach of river can not occur until it has happened at the ice control for the reach.

The series of events leading up to the fracturing and subsequent flushing of the river ice on long reaches of large rivers is not the same as on the small rivers with short reaches. Consequently, the two can not be lumped together in hopes of finding an "universal function" describing something called "breakup". On the rivers with long reaches between successive ice controls the positive surge from an upstream release can, under the right condition outrun the snowmelt hydrograph. When this occurs, the properties of the positive surge become the primary factors in forecasting the occurrence of an "ice out" condition at a given location on a river. Practitioners involved with river forecasting are well aware of such conditions. For example, the Alaska River Forecast Center, NWS, routinely forecasts the occurrence of "ice out" condition at various locations along the Yukon and Kuskokwim based solely on a time lag from an upstream station (i.e. the celerity of the positive surge).

Under certain conditions associated with some breakups the ice cover initially begins to move with the flow then jams in some horizontal constriction. The initiation of the breakup process is identical to the model presented above. The positive and negative surges mentioned in the normal breakup must also exist with the horizontal constraint or jam situations. If the ice floes associated with the progression of the positive surge encounter some blockage which retards their movement, then from a hydraulic standpoint their stalling is identical to the stalling of the initial ice floes associated with freeze-up. The unsteady, non-uniform flow model developed for freeze-up is basically the same as that which must exist with the ice jam. The downstream edge of the jam forms the new temporary ice control. The same basic types of surface profiles will develop upstream of the jam. However, in most cases the roughness of the ice associated with an ice jam is greater than that associated with freeze-up and consequently greater stage changes will occur. If the hydraulics of the system governs, i.e. the strength of the jam is greater than the force exerted by the flow, then the jam will persist until the stage at the ice control, GH_B, is sufficient to allow the floes to clear the horizontally imposed construction. Once that condition is met, the hydraulics is identical to the conventional breakup model given above.

Effects of Section Geometry and Energy Gradeline Slope

The hydraulic response of virtually all reaches of all rivers is believed to be the same as given above. However, the magnitude of what appears to happen is highly dependent upon the geometry of both the section and the reach of river. For the wide shallow section and the most efficient stable alluvial section with cohesionless soils, the added resistance caused by an ice cover will result in an increase in flow depth of approximately 30% for for the uniform flow section - all other factors remaining constant. However, if the soil conditions change along the length of a river such that the resulting section geometry changes to a deep, narrow section, the increase in flow depth resulting from the added resistance of the ice may drop to as low as 10% - all other factors remaining the same. If the flow is non-uniform and accelerating, i.e. an M-2 profile, the percentages cited above may be considerably less.

When an ice cover forms on a reach of river, the slope of the energy gradeline immediately upstream of the section control steepens and the length of the M-2 profile increases over what had existed for the open water condition. For the deep section with flat slope, a relatively small change in depth along some given length is required to produce the necessary percentage change in slope. A very long profile with a relatively small amount of storage develops. Consequently, the depths y_0 & y_2 of Figure 3 will be nearly the same. With the initiation of breakup, the propagation of the resulting waves is very weak. To the casual observer, the ice cover merely appears to float up with the rising stage and float downstream.

APPLICATION OF THE MODEL

The model has been applied to a number of streams in Michigan's Upper Peninsula. The results from two of these are included for illustrative purposes.

The Brule River near Florence, Wisconsin is typical of the Type II response cited above. The gage is located 1.6. km upstream of the confluence with the Paint River. At times a slight increase in stage (1-2 cm) is seen as a prelude to breakup. However, at breakup the stage merely drops 1.3 to 1.5 m (4 to 5 ft) in less than 2 days. A similar condition has been noted to occur each and every year. A copy of a working hydrograph prepared by the USGS is shown in Figure 4*. The upper curve is the equivalent open water discharge based on the indicated stage. The dashed line is the USGS estimate of under ice discharge. It will be noted that the stage at break- up is less than the maximum stage at freeze-up. A similar condition occurs in the vast majority of cases examined.

A similar hydrograph for the Sturgeon River near Nahma Junction, Michigan is shown in Figure 5. This stream is typical of what was classified as a Type I response. The stage associated with the flow condition of early April set the elevation of the shore fast ice (GH_w). On 10 April the corresponding stage was 4.95 ft. Three days later, i.e. 13 April, a routine flow measurement was made by the USGS and the average ice thickness found to be 1.4 ft. As noted on the hydrograph the initiation of breakup occurred on 17 April at an indicated stage of 6.41 ft. The model of equation 2 would have indicated a breakup stage, GH_B, of 6.35 ft which is in very close agreement with the observed values.

To date the data for 5 consecutive winters on the Sturgeon River at Nahma Junction have been examined. In 4 of the 5 cases the breakup stage indicated by equation 2 was within ± 0.2 ft of the observed value - well within the natural variability of the ice thickness measurements. On the one remaining year, there was a very gradual runoff and the ice merely "rotted" in place. In all cases, the stage associated with breakup was less than the maximum stage associated with freeze-up. (According to the model proposed by Beltaos (1983), the stage at breakup must be greater than the maximum stage at freeze-up).

The model of equation 2 has also been successfully applied to 2 other rivers in Michigan's upper peninsula. There, as with the data for the Sturgeon River at Nahma Junction, the variability of the ice thickness is greater than the difference between the predicted and observed stage at breakup. For the 5 year period of record that has been examined to date for both the Manistique River near Manistique and the Sturgeon River near Sidnaw (a different Sturgeon River than that cited above), the stage at

* Note: The USGS used the English system of units in its measurements of flowrate. To be consistent with their work the hydrographs are shown in the English system. $1 m^3/s = 35.31$ cfs.

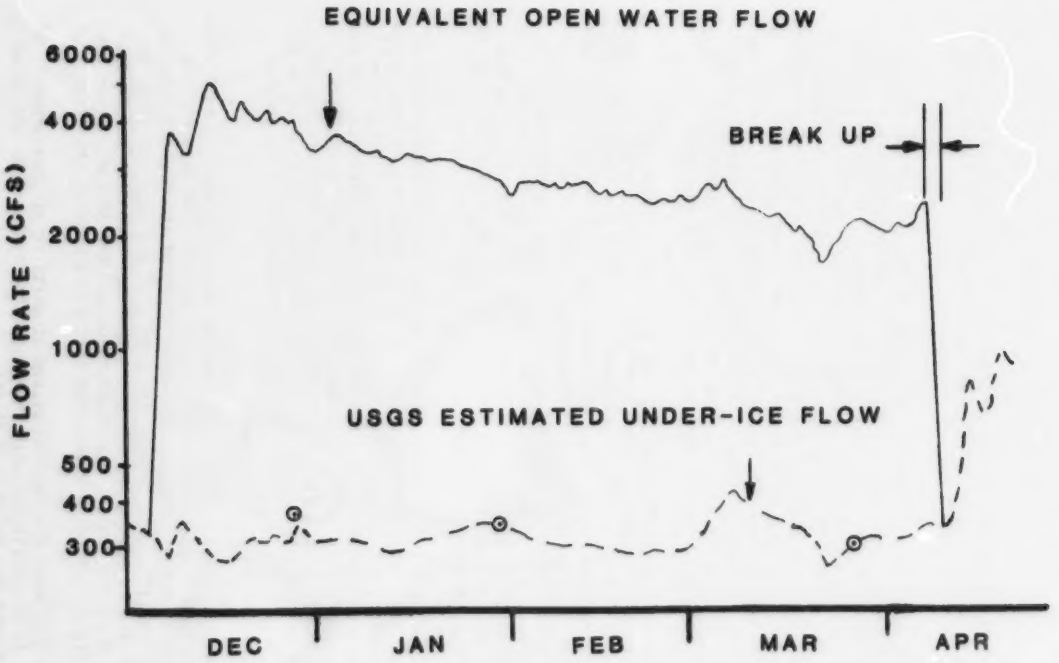


Figure 4. Example of Type II breakup. Brule River near Florence, Wisconsin. WY 1974.

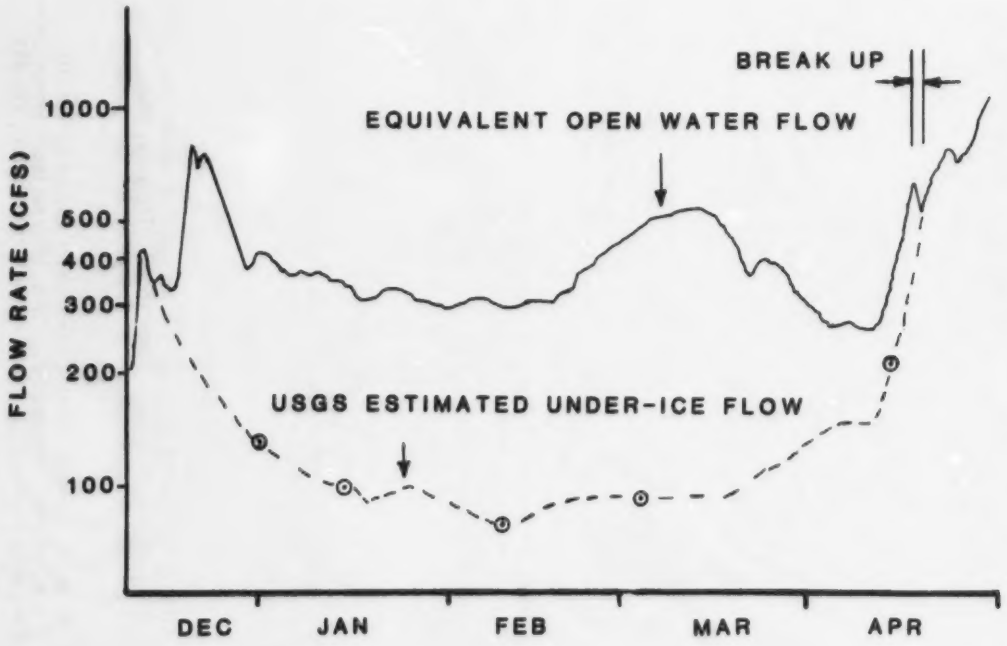


Figure 5. Example of Type I Breakup. Sturgeon River near Nahma Junction, Michigan. WY 1972.

breakup was found to be at times above, and at other times below the maximum stage at freeze-up. The controlling factor appears to be the elevation of the shore fast ice just prior to the spring runoff and the thickness of the ice. The response of the stage at freeze-up is the result of a completely different hydraulic process than what occurs at breakup as defined above.

SUMMARY AND CONCLUSIONS

Previous studies have attempted to relate the entire process of the fracturing and subsequent flushing of an ice cover into a broad term called "breakup". The initial movement of the ice cover, the behavior of ice jams, and the effect of surges have all been lumped together. The hydraulics of each of these phenomen associated with the "ice out" condition are distinctly different processes and can not be lumped together for a universal analysis.

Here, "breakup" is defined as the unsteady, non-uniform flow condition associated with the transition from fixed ice cover to semi-free surface flow under an acceleration mode. The initiation of breakup is defined as the instant in time when the flow first changes from steady to unsteady conditions. Breakup is completed when the flow can again be analyzed using the steady state assumptions.

With the above definition, a model is proposed that relates the stage at breakup, GH_B, to the stage just prior to the spring runoff, GH_V, and the ice thickness. The model has been successfully applied to several streams located in Michigan's upper peninsula. However, much additional research is necessary to incorporate possible regional variations into the observed response.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the cooperation and assistance of the U.S. Geological Survey and in particular the staff of the subdistrict office, Escanaba, Michigan.

REFERENCES

Beltaos, S., (1983), Initiation of River Ice Breakup. Proceedings: 4th Northern Research Basins Symposium Workshop, Ullensvang, Norway, March 22-25, 1982. Norwegian National Committee for Hydrology, Oslo, 163-177.

Henderson, F. M. and R. Gerard, (1981), Flood Waves Caused by Ice Jam Formation and Failure. Proceedings: IAHR International Symposium on Ice, Quebec, July 27-31, 1981. Universite Laval, Quebec, 209-219.

Kent, T., et. al. (1984), Ice Regime Reconnaissance, Yukon River, Yukon. Proceedings: 3rd International Specialty Conference on Cold Regions Engineering, Edmonton, April 4-6, 1984, ASCE/CSCE, Vol. 3 (in publication).

Santeford, H. S. and G. R. Alger, (1984a), The Hydraulics of River Ice - A Summary. Proceedings: 5th Northern Research Basins Symposium Workshop, Vierumaki, Finland, March 19-23, 1984. National Board of Waters, Helsinki, 3.25 - 3.55.

Santeford, H. S. and G. R. Alger, (1984b), Predicting Flowrates in an Ice Covered Stream. Proceedings: 3rd International Specialty Conference on Cold Regions Engineering, Edmonton, April 4-6, 1984, ASCE/CSCE, Vol. 3 (in publication).

Santeford, H. S. and G. R. Alger, (1984c), The Hydraulics of Freeze-up. Proceedings: Specialty Conference Hydraulics Division, ASCE, Coeur d'Alene, Aug. 14-17, 1984 (in publication).

DISCUSSION

Richard Penn

In one of our rivers, we notice in some years a drop in stage. In other years, we notice stage increases. This appears to be due to how fast the freshet arrives: if slowly, then you should notice the negative wave, if quickly, you won't.

Reply

When breakup occurs, the negative wave must occur. However, if the local flood (runoff) hydrograph is increasing very rapidly, then the combined effects of the flood hydrograph plus the negative wave tend to cancel each other if the flood wave is sufficiently strong, then it tends to make the negative wave - all that the gauge "sees" is the rise.

SESSION C

ICE JAMS

Chairman: Mr. D. Andres; Alberta Research Council

SESSION C

EMBACLES DE GLACE

Président: M.D. Andres; Counsell de recherche de l'Alberta

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

CASE STUDIES CONCERNED WITH ICE JAMMING

Sylvester Petryk
Rousseau, Sauvé, Warren, Inc. - Montreal
for

Working Group on River Ice Jams*,

NRCC Subcommittee on Hydraulics of Ice-covered Rivers

Abstract

On many rivers experienceing ice jams, studies have been carried out by various entities concerned with planning, design, construction and operation of river development projects. The NRCC Working Group on River Ice Jams has collected descriptions of 71 case studies from various authors. Relevant topics described include amongst others: field data collection programs; analysis of field ice jam conditions; ice jam control measures at specific sites; evaluation of backwater conditions in the presence of ice jams by means of mathematical models; and application of physical models. This paper gives in tabular form a very brief description of each case study identified. The tabular description gives the title and river location of the case study, field conditions studied and/or observed, and the person(s) to contact if further information is desired.

---

* Members of the NRCC Working Group on River Ice Jams:

S. Beltaos (Chairman)	R. Gerard	S. Petryk
D. Andres	R.A. Halliday	T. Prowse
B. Burrel	F. Parkinson	

The paper is designed to increase the exchange of ice jam related field data and results of studies among ice specialists and engineers. Informal publications as well as more publications of formal technical articles are being encouraged. Four different formats are suggested to potential future contributors to help promote this exchange of ice jam information.

Sommaire

Des études ont été entreprises par divers organismes engagés dans la planification, la conception, la construction et l'opération des aménagements hydriques. Le Groupe de Travail du CNRC sur les Embâcles des Glaces* a recueilli les descriptions de 71 études de cas provenant de divers auteurs. Les sujets d'intérêt comprennent entre autres: programmes de relevés sur terrain; analyse des relevés des glaces; contrôle des embâcles à des sites spécifiques; évaluation des courbes de remous en présence d'embâcles à l'aide des modèles mathématiques; application des modèles physiques. Cet article comporte une brève description sous forme tabulaire de chaque étude de cas identifié. La description tabulaire fournit le titre de l'étude et l'emplacement de la rivière, les conditions physiques étudiées et/ou observées, et la (les) personne(s) à contacter si des renseignements additionnels sont désirés.

Cet article a été conçu dans le but de stimuler les échanges de données relatives aux relevés des embâcles et les discussions portant sur les résultats d'études effectuées parmi les spécialistes et les ingénieurs dans le domaine des glaces. La publication des articles formels ou informels est encouragée. Quatre formats différents sont suggérés aux auteurs éventuels dans le but de promouvoir l'échange de renseignements sur les embâcles des glaces.

1.0 INTRODUCTION

The NRCC Working Group on River Ice Jams was established in 1982 to help improve the available technology for dealing with ice jams. Task number 4 of the Working Group is concerned with documentation and publication of case studies concerned with ice jams. In order to help carry out this task, about 35 persons were contacted from different organizations in an effort to identify as many case studies as possible. As a result, 65 case study descriptions have been received prior to publication time of this paper. These descriptions give an outline of case studies proposed, underway, or completed. Reference (1) gives a compilation of the case study descriptions received by the Working Group.

This paper presents the case study descriptions in summarized form in Table 1; each case study is described with the following information:

- Case number assigned for reference purposes.
- Title of case study and location of river.
- Field conditions studied and/or observed.
- Person to contact if more information is desired.

The methodology for encouraging future publication of case studies is given in section 3.0..

2.0 ICE JAM TOPICS TREATED BY THE CASE STUDIES

A wide range of ice jam topics are covered amongst the case study descriptions summarized in Table 1. The following subjects are treated, amongst others:

- Thermal heat balance in a river during freeze-up and spring break-up;
- Freeze-up ice jamming and ice cover progression
- Spring break-up and ice jamming
- Winter break-up and ice jamming
- Ice thickness measurements
- Design criteria for ice regime calculations (natural and regulated river flow conditions)
- Documentation and studies of past ice jam flooding events

- Ice jam effects on river bank and bed erosion
- Analysis and observation of hydraulic transients created by relatively sudden ice jam release
- Analysis, control and observations of ice regime upstream and downstream of existing hydro-electric works
- Ice studies and field surveys for future hydro-electric works
- Effects of hanging dams on break-up and ice jamming
- Numerical models of river ice regime (developments, calibrations, and applications)
- Physical modeling of river ice regime
- Prevention and control of ice jams by means of weirs, islands, ice booms, excavations, flow and level regulation, cutting slots, ice removal, and blasting
- Wind and lake seiche effects on ice jamming
- Hydrometeorological effects on break-up and ice jamming
- Documentation of damages due to flooding caused by ice jams
- Floodplain mapping studies including ice jam effects
- Engineering studies concerned with ice jam control measures and structures for hydro-power and flood control projects
- Surveys of ice blockages in powerhouse approach canals
- Aerial and ground surveys of ice jams (locations, profiles, and affected reaches)

3.0 FUTURE PUBLICATIONS OF CASE STUDIES CONCERNED WITH ICE JAMS

3.1 Types of publications:

In the future, the NRCC Working Group on River Ice Jams will be encouraging and facilitating the publication of additional case studies according to the following four formats:

a) Short descriptions of case studies presented individually on about 1 to 10 pages in length. These will be published in loose leaf form in the "Casebook on Ice Jams". Periodically the casebook would be updated with new contributions from researchers and practitioners working in the ice jam field of interest. The

recommended format for these short descriptions of case studies is described below in section 3.2.

b) Technical articles presented in semi-formal format to be presented at technical meetings. Publication in the proceedings of the NRCC Workshops on the Hydraulics of River Ice will be particularly encouraged.

c) Publication through the NRCC Working Group of formal and informal reports on case studies giving a detailed description of field data collected and/or a complete description of the results from a study on a given river.

d) Publication of formal technical articles in well known hydraulics and river ice publications such as the Canadian Journal of Civil Engineering, ASCE Journal of Hydraulic Engineering, International Symposium on Ice, etc.

3.2 Recommended Format for Contributors to Future "Casebook on Ice Jams"

Short descriptions of case studies on ice jams will be classified according to the regional location of each river (Yukon and Northwest Territories, Atlantic Canada, Central Canada, Western Canada, "Rivers located in more than one region of Canada", or "Rivers Outside of Canada"). Therefore, each case study should be presented individually, normally varying in length from one to about ten pages. The following format is preferred, although alternate formats will be considered for publications.

1) Title
2) Name and location of river
3) Scope of work
4) Status of work
5) References (if any)
6) Contact(s)
7) Date of submission, and person contributing the case study description (if different from the contact person).

Most of the descriptions of case studies given in reference (1) will be included in the "Casebook on Ice Jams". Case study descriptions should be submitted to:

S. Petryk
Rousseau, Sauvé, Warren, Inc.
1155 Metcalfe Street
Suite 840 - Sun Life Building
Montreal, (Quebec)
H3B 2V6

or

S. Beltaos
Hydraulics Division
National Water Research Institute
867 Lakeshore Road
P.O. Box 5050
Burlington, Ontario
L7R 4A6

4.0 CONCLUDING COMMENTS

Most parts of Canada experience an important degree of ice jamming on their rivers. Because of the complexity and limited knowledge of ice jams, reporting of field observations and ice jam related studies on particular rivers is very important to both researchers and engineers. For this reason the Working Group on River Ice Jams, by means of this paper and reference (1), has published a list of case studies and contact person(s) to promote the exchange of ice jam related information.

In the future the Working Group plans to continue encouraging publication of case studies according to the methodology outlined in this paper. New ideas, submitted to any one of the Working Group members, would be appreciated.

5.0 ACKNOWLEDGEMENTS

The tables presented in this paper were prepared from case study descriptions given in reference (1). The Working Group would like to express their sincere appreciation to each of the contributors of the case study descriptions, and to Rousseau, Sauvé, Warren Inc. for supporting the publication of this paper.

6.0 REFERENCE

1) Petryk, S. (editor for the NRCC Working Group on River Ice Jams): "Case studies Concerned with Ice Jamming: A compilation of Descriptions from Contributors". A working report presented at the 3rd Workshop on the Hydraulics of River Ice, June 20-21, 1984, Fredericton, N.B.

TABLE 1: LIST OF CASE STUDIES CONCERNED WITH ICE JAMMING

Case Number	Title of Case Study and Location of River	Field Conditions Studied and/or Observed	Contact
	Rivers in Yukon and Northwest Territories		
T101	Ice Regime of the Lower Mackenzie River and Mackenzie Delta (NWT)	Freeze-up, pre-break-up ice thicknesses, and break-up	1) J.C. Anderson National Hydrology Research Institute (NHRI) - Ottawa
T102	Liard and Mackenzie River Ice Break-up (Fort Simpson, N.W.T.)	Break-up and ice jams	T.D. Prowse or J.C. Anderson NHRI - Ottawa
T103	Historical Flood Review (Mackenzie River Basin)	Flooding due to ice jams during spring break-up	J.C. Anderson or T.D. Prowse NHRI - Ottawa
T104	Ice Jams and Break-up on the Yukon River (Dawson City, Yukon)	Break-up and ice jam floodings	1) S.C. Bigras, NHRI, Ottawa 2) J. McLachlan Indian and Northern Affairs Canada Whitehorse, Y.T. 3) R.O. Lyons Inland Waters Directorate (IWD) Vancouver, B.C.
T105	Flood Risk Mapping (Hay River, NWT)	Break-up and ice jams, design crateria for ice regime	Dept. of Indian and Northern Affairs - Yellowknife, NWT
T106	Spring Break-up Observations at Pipeline River Crossings in the Keewatin District (Thelon and Kazan Rivers, NWT)	Effects of break-up on river bank erosion	J.C. Anderson NHRI - Ottawa
T107	Mackenzie River Near Arctic Red River (NWT)	Observation of ice jam release	C.R. Neil or R.H. Cooper Northwest Hydraulic Consultants Edmonton
T108	Mackenzie River at Fort Simpson	Ice jam observations including water level measurements	1) F.E. Perkinson LaSalle Hydraulics Laboratory Montreal 2) L.J. Parmley, B.C. Hydro Vancouver, B.C.
T109	Mackenzie River at Norman Wells	Breakup and ice jamming	Same as for Case T108
T110	Mackenzie River at entrance to Mackenzie Delta	Massive ice jams	Same as for Case T108
T111	Liard River at Beaver Dam	Freeze-up ice jamming	Same as for Case T108
T112	Liard River at Fort Liard	Freeze-up ice jamming	Sale as for Case T 108

TABLE 1: LIST OF CASE STUDIES CONCERNED WITH ICE JAMMING (Continued)

Case Number	Title of Case Study and Location of River	Field Conditions Studied and/or Observed	Contact
	Rivers in Western Canada		
T201	Freeze-up Jamming and Effectiveness of Ice Anchor on the Reduction of Flood Levels on the Bow River, Alberta	Freeze-up ice jamming	1) D. Andres Alberta Research Council Edmonton 2) G. Forstad Alberta Environment, Edmonton
T202	Spring Melt and Break-up in a Central Alberta Basin	Ice jamming at freeze-up and	Same as for Case T201
T203	Freeze-up on the North Saskatchewan River at Edmonton, Alberta	Observation and analysis of freeze-up	1) D. Andres Alberta Research Council, Edmonton 2) R. Gerard University of Alberta, Edmonton
T204	Freeze-up Jams and Associated Water Levels on the North Saskatchewan River at Rocky Mountain House, Alberta	Observation and analysis of freeze-up downstream of power dam	1) P. Doyle Trans Alta Utilities Corp. Calgary 2) D. Andres Alberta Research Council, Edmonton
T205	Ice Related Problems on the Peace River at the Town of Peace River, Alberta	Freeze-up, break-up, and ice jamming	Same as for Case T201
T206	Break-up and Ice Jams on the Athabasca River at Fort McMurray, Alberta	Profiles of ice jams, and surges resulting from ice jam failure	1) D. Andres Alberta Research Council, Edmonton 2) H. Rickert Alberta Environment, Edmonton
T207	The Ice Regime of the Peace River in the Vicinity of Portage Mountain Development, Prior to and during Diversion (B.C.)	Formation and break-up of hanging dams and ice jams	F. Sampson B.C. Hydro - Vancouver
T208	Observation and Analysis of Freeze-up Jams on the Peace River near Taylor (B.C.)	Observation and numerical modeling of freeze-up ice jams	C.V. Karthe or T. Keenhan B.C. Hydro - Vancouver
T209	Peace River at Town of Peace River (Alberta)	Ice Jam water levels associated with fluctuating hydro plant releases	1) C.R. Neil Northwest Hydraulic Consultants 2) D.D. Andres Alberta Research Council, Edmonton
T210	Nelson River at Long Spruce Project	Ice jam development accompanied by massive shove	1) F.E. Parkinson LaSalle Hydraulic Laboratory Montreal 2) P.D. Wang Manitoba Hydro, Winnipeg 3) R. Carson, Crippen-Acres Winnipeg, Manitoba

TABLE 1: LIST OF CASE STUDIES CONCERNED WITH ICE JAMMING (Continued)

Case Number	Title of Case Study and Location of River	Field Conditions Studied and/or Observed	Contact
	Rivers in Central Canada		
T301	Rivière des Milles Îles - Étude du régime des glaces (Québec)	Analysis of ice regime and identification of methods to eliminate flooding	A. Carpentier J. Monfet R. Poulin Ministère de l'Environnement Sainte-Foy, Québec
T302	Étude du régime des glaces pour solutionner des problèmes d'inondation à Richmond (Québec)	Ice observations to calibrate mathematical model and examination of measures to reduce flooding	G. Barbé Ministère de l'Environnement Sainte-Foy, Québec
T303	Construction d'une estacade sur la rivière Saint-Charles (Québec)	Ice Boom and weir constructed to control ice regime	A.-R. Tremblay and A. Valcourt Ministère de l'Environnement Sainte-Foy, Québec
T304	Mathematical model RIVHER (Québec)	Mathematical model of river ice and thermal regime applied to Quebec rivers	N. Marcotte T. Quach Hydro-Québec - Montréal
T305	Richelieu River between Chambly and Lake Champlain	Observations and simulation of of river ice regime	Same as for Case T304
T306	Chaudière River at Charny (Québec)	Ice blockage of approach canal to power house	N. Marcotte Hydro-Québec - Montréal
T307	Unpublished, Ice Jam Events of 1979, 1981, 1982 and 1984, Thames River, Ontario	Observation, analysis of field data, and computation of ice jam profiles	W.L. Knowles MacLaren Engineers Inc. London, Ontario
T308	St. Lawrence, International Rapids Section (Ontario)	Prevention of important ice jams by excavations, ice booms and flow control	D.M. Foulds Uxbridge, Ontario
T309	Niagara River, Lake Erie to Lake Ontario	Control of ice jamming by flow control, excavations, and ice booms. Effects of lake seiche on ice regime.	Same as for Case T308
T308	Madawaska, Ontario	Effects of hydro releases on ice regime	Same as for Case T308
T311	Mississagi River, Northern Ontario	Effects of hydro releases on downstream flooding with ice effects	Same as for Case T308

TABLE 1: LIST OF CASE STUDIES CONCERNED WITH ICE JAMMING (Continued)

Case Number	Title of Case Study and Location of River	Field Conditions Studied and/or Observed	Contact
	Rivers in Central Canada (continued)		
T312	Kawinistikwa River, North Western Ontario Thunder Bay	Hydrometeorological effects on ice jamming	Same as for Case T308
T313	Town Creek in Timmins, Ontario	Aufreis flooding	Same as for Case T308
T314	Aubrey Falls Generating Station Winter Peaking Restrictions (Ontario)	Effects of powerhouse operation on ice jams resulting from broken ice and frasil	T.J. Wigle Ontario Hydro - Toronto
T315	Ice Cover Collapse on the St.Lawrence River	River ice observations and ice cover thickness measurement. Control of ice regime with powerhouse operation	Same as for Case T314
T316	Port Bruce Preliminary Engineering Study (Ontario)	Assessment of flood damages with and without ice jams. Feasability of flood control measures	H. Belore Cumming-Cockburn & Assoc. Willowdale, Ontario
T317	Brieze Dam Ice Control Capability (Ontario)	Assessment of ice jam flooding problem with the help of a mathematical model	Same as for Case T316
T318	Lower Credit River Ice Control Study (Ontario)	Assessment of ice jamming and identification of remedial measures	Same as for Case T316
T319	Lonedale Dam (Ontario)	Identification and analysis of ice jam problems with the help of a mathematical model	Same as for Case T316
T320	Lott Dam/College Street Dam (Ontario)	Effects of six dams on the ice regime on the Moira River	John Jutts Cumming-Cockburn & Assoc. Willowdale, Ontario
T321	Criteria for Stability of Ice Covers in Rivers (St.Lawrence River, Ontario)	Observations and analysis of ice jamming. Comparison of theory with observations	D. Witherspoon Great Lakes St.Lawrence Study Office Cornwall, Ontario
T322	Ice Management on the Rideau River, Ottawa	Ice Jam control measures	D.G. Curry City of Ottawa Ottawa, Ontario

TABLE 1: LIST OF CASE STUDIES CONCERNED WITH ICE JAMMING (Continued)

Case Number	Title of Case Study and Location of River	Field Conditions Studied and/or Observed	Contact
	Rivers in Central Canada (continued)		
T323	Field Studies of River Ice Jams and Flooding (Themes River and Grand River, Ontario)	Documentation and analysis of freeze-up, break-up and ice jamming	S. Beltaos National Water Research Institute Burlington, Ontario
T324	Ice Considerations in the Design of Lac Robertson Hydro-Electric Plant (Québec)	Analysis of solid ice cover break-up, ice jamming and melting effects	S. Petryk Rousseau, Sauvé, Warren Inc. Montreal
T325	Ice Considerations at LG2-PA Powerhouse Expansion (Québec)	Evaluation of ice regime in power plant intake canal	Same as for Case T324
T326	St.Lawrence River Ice Studies for Harnessing the Lachine Rapids in Montreal	Frazil and ice jamming effects on the design and operation of hydro-electric works	R.J. Silver Rousseau, Sauvé, Warren Inc. Montreal
T327	Observations of Ice Conditions on the St.Lawrence River (Ontario and Québec)	Documentation of ice regime by various entities	1) Ontario Hydro 2) Hydro-Québec 3) Saint Lawrence Seaway Authority 4) International Power Commission
T328	Ice Management - Moira River	Assessment of ice-related flooding, and design of capital works to control it	1) J. Kelleher Moira River Conservation Authority Belleville, Ontario 2) N. Beasley Kilborn Engineering Ltd Toronto
T329	Chateauguay River, at mouth, Lake St.Louis	Proposed ice jam study site	F. Perkinson LaSalle Hydraulic Laboratory Montreal
	Rivers located in Atlantic Canada		
T401	Environmental Assessment of Dickey-Lincoln Project (N.B.)	Cyclical ice formation, break-up and jamming downstream of power dam	U.S. Army Corps of Engineers New England Division Waltham, Mass.
T402	Walker Brook Hydrotechnical Study Canada, N.B. Flood Damage Reduction Program	Effects of debris and ice jams on water levels	H. Belore Cumming-Cockburn & Assoc. Willowdale, Ontario
T403	Hydrotechnical Study of Stephanville Area Canada, Newfoundland Flood Damage Reduction Program	Comparison of flood levels with and without ice and debris jams. Simulations with HEC-Ice and NWRI empirical procedures	Same as for Case T402
T404	On-going Field Observation Program (N.B.)	Ice cover thicknesses, qualitative ice regime descriptions, ice jam locations etc.	B.C. Burrell Environment New Brunswick Fredericton, N.B.

TABLE 1: LIST OF CASE STUDIES CONCERNED WITH ICE JAMMING (Continued)

Case Number	Title of Case Study and Location of River	Field Conditions Studied and/or Observed	Contact
	Rivers Located in More than One Region of Canada		
T501	Liard - Mackenzie Winter Regime Study (B.C. and NWT)	Effects of hydro power operation on ice regime. Ice regime observations. Development and application of mathematical model.	L.J. Parmley B.C. Hydro Vancouver
T502	River Ice Break-up (Fort Nelson, B.C. and Lower Liard River, NWT)	Ice thicknesses, break-up, location and severity of ice jams.	T.D. Prowse or J.C. Anderson NHRI - Ottawa
T503	Annotated Bibliography on River Ice Regimes	Freeze-up, ice jams, break-up, river regulation effects on ice regime, etc.	J.C. Anderson NHRI - Ottawa
T504	Experience with the Numerical River Ice Model "SIMGLACE"	Development and applications of numerical river ice model.	S. Petryk Rousseau, Sauvé, Warren Inc. Montreal
	Rivers Located Outside of Canada		
T601	Observation of River Ice Break-up on Alaska Rivers	Aerial photos of ice break-up, ice and water measurements to develop mathematical model of break-up	T.E. Osterkamp J.P. Goelnk University of Alaska Fairbanks, Alaska
T602	Gunison River Ice Jams	Variables affecting ice jamming	P. Burgi Bureau of Reclamation Denver, Colorado
T603	Strategic Hydro-Power Operation at Freeze-up Reduces Ice Jamming	Effects of hydro operations and engineering works on ice cover formation and ice jamming	L.H. Bilfalk Swedish State Power Board Sweden
T604	Advanced Remedial Measures - Ice Jam, Montpelier, VT, 19 January 1978	Ice jam flooding and its control	D. Calkins U.S. Army, Cold Regions Research and Engineering Laboratory (USA CRREL) Hanover, N.H.

TABLE 1: LIST OF CASE STUDIES CONCERNED WITH ICE JAMMING (Continued)

Case Number	Title of Case Study and Location of River	Field Conditions Studied and/or Observed	Contact
	Rivers Located Outside of Canada (continued)		
T605	Ice Conditions on Missisquoi River at Richford, VT, August and October 1980	Break-up and ice jam flood	Same as for Case T604
T606	Misc. Documents on Hardwick Ice Jam Flooding 1974 to present	Ice Jam Flooding	Same as for Case T604
T607	Detailed Project Report - Whitesboro, NY	Fluvial and ice jam flooding study	Same as for Case T604
T608	Ottauquechee River Ice Studies	Winter regime documentation and physical modeling	Same as for Case T604
T609	Detailed Project Report - Binghamton, NY, 1982	Ice jam flooding and control measures	Same as for Case T604
T610	Ice/Hydraulic Measurements - Salmon River, Idaho	Freeze-up and hydraulic data for flooding study	Same as for Case T604
T611	Israel River Ice Impoundment Weir	Ice control structure to prevent ice jam flooding	Same as for Case T604
T612	Oil City Ice Control Structure - 1983	Ice control structure to control freeze-up and ice jam flooding	D. Deck USA CREEL Hanover, N.H.

DISCUSSION

Nabil Elhadi, Chief, Surface Water Section, N.B. Environment.

I have the understanding that you have concentrated on unpublished case studies. However, there are various publications of case studies which would be of interest to the working group. Was there an established criteria to go this route?

Response: The term "published case studies" refers to case studies on ice jams which are published in well known publications such as the Canadian Journal of Civil Engineering, ASCE Journal of Hydraulic Engineering, and the International Symposium on Ice. The reason that the Working Group has concentrated on unpublished case studies is that the published case studies are already available to interested persons through librairies and bibliographic search services such as the Canadian Institute for Scientific and Technical Information.

However, it is recognized that a bibliography of published case studies on ice jams would be a useful reference. As a result the Working Group is currently considering the possibility of adding a bibliography of case studies on ice jams to the "Casebook on Ice Jams" described in this paper.

Dale I. Bray, Professor of Civil Engineering, University of New Brunswick

Will you be presenting a few caracteristic values associated with the ice jam for each case study; such as drainage area, discharge at break-up, mean annual discharge?

Response: Some of the caracteristic values which you mentioned are presented in the descriptions given in reference (1). In the future the Working Groups will be publishing similar descriptions in the "Casebook on Ice Jams". It will be a sort of ice jam source catalogue from which interested users can determine relevant studies, reports and contacts from which more detailed informations can be obtained.

The Working Group does not presently intend to synthesize this information by presenting caracteristic values for each case study as you have suggested.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21,1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

CUTTING TRENCHES IN AN ICE COVER TO PREVENT ICE JAMS

L. Jolicoeur[1], B. Michel[2] et J. Labbé[3]

ABSTRACT

This paper deals with an experience of ice cutting to prevent ice jams in the Beaurivage river (near Quebec city) during the 1983 spring break up.

A reach 600 m long, prone to ice jams, had been opened up by explosives the year before so that a comparison could be established between the two methods.

Five types of ice patterns were made with a Case "DH4" trencher and the results for each one are discussed in the paper. In order to do this work, a field study had to be done on the bearing capacity of weakened ice and its ability to carry the load of the machinery just prior to break up.

Finally, observations were made from an helicopter so that a film was taken showing the ice cover just before, during and after break up of the ice in this river reach.

It was found that this method, with simple trenching patterns, was more effective, cheaper and more environmentally acceptable than the blasting of the previous year.

[1] Recherches B.C. Michel, Inc. Now with "Roches Associés Ltée"

[2] Recherches B.C. Michel, Inc.

[3] Ministère de l'Environnement, Québec

RÉSUMÉ

Cet article traite d'une expérience de découpage de la glace, comme mesure préventive contre la formation d'embâcles, sur la rivière Beaurivage (près de Québec) au printemps 1983.

Un tronçon d'une longueur de 600 m, prédisposé à la formation d'embâcles, avait été le site d'une expérience sur le dynamitage l'année précédente. Ainsi une comparaison a pu être faite entre les deux méthodes.

Cinq différents patrons de tranchées ont été réalisés à l'aide d'une machine à tranchées "DH4" de la compagnie Case. Les résultats de chaque patron sont discutés dans cet article. Préalablement, une étude de la capacité portante du couvert de glace (glace affaiblie) avait été faite. Cette étude détermina le poids maximum de la machine à trancher la glace qui fut utilisée.

Finalement la rupture du couvert de glace a été observée à partir d'un hélicoptère. Un film a été tourné montrant le couvert de glace juste avant, pendant et après la rupture de la glace dans le méandre.

En résumé cette méthode, avec de simples patrons de tranchées, a démontré qu'elle était plus efficace, moins coûteuse et moins dommageable pour l'environnement que le dynamitage des années précédentes.

INTRODUCTION

Ice jams in Quebec rivers are causing, every spring, important floods and severe damages to shore properties.

The cutting of trenches in the ice cover to prevent ice jams has never been completely studied, and only rectilinear trenches with or without explosives have been used. The 1983 spring experience, done by the Quebec Ministry of the Environment, had for objective to find a new method to prevent ice jams, which would be cheaper but above all more environmentally acceptable than the use of explosives.

After a litterature review we will present the different steps of the experiment. We will finish with a comparison between this method and blasting.

LITTERATURE REVIEW

Bolsenga (1968) and Michel (1971) discuss at length various methods used to prevent ice jams. They are ice breaking, dusting, the use of air curtains, thermal regime modifications, explosives, ice control structures and channel modifications. Unfortunately nothing is said for the method of cutting trenches in the ice cover to weaken it prior to break up.

Nevertheless the cutting of trenches in the ice cover is an old method used to prevent ice jams and reduce flooding.

At the beginning trenches were made with handsaws. Ice blocks were retrieved to prevent the trenches from freezing again.

The cutting of keys in the ice cover, jointly with blasting has been used since 1970 in the Rideau river near Ottawa. The keys are cut and spaced at a distance of one hundred feet in the same direction as the main flow and approximately fifty feet from each shoreline (Deugo, 1973).

The cutting of trenches seems to be a method to prevent ice jams which had never been studied in a comprehensive manner.

RIVER CHARACTERISTICS

The Beaurivage river is located near Quebec city on the south shore of the St. Lawrence river. This river is flowing from south to north which direction is more prone to ice jams formation.

The experiment site (Figure 1) is a meander, 600 m long, followed in the lower part by a rectilinear section. This portion of the river has a mean width of 36 m and a low velocity flow.

At the time of ice trenching to mean water depth and flow discharge were respectively 2,0 m and 3 m^3/s.

BEARING CAPACITY OF ICE COVER

A field study was realized in order to determine the bearing capacity of rottened ice. The study was needed to make the choice of a "trencher".

The ice and snow depth were measured along five transversal sections, every 5 meters. For each section a block of ice was retrieved for crystallographic analysis. Supplementary holes were made at critical points.

The ice cover had a minimal thickness of 50 cm with an average of 25 cm of snow ice (T_1) and 25 cm of columnar ice (S_2). The snow cover had a varying thickness from a few cm to 30 cm.

The ice cover was well attached to the river banks and no major cracks were observed at the time of the measurements.

It was found that the ice cover would permit a maximum load of 2500 kg (this load took into account the presence of a trench and rotten snow ice). The influence zone of a 2500 kg load on the 50 cm thick ice cover has a 8 m radius. This signifies that two 2500 kg loads had to be separed more than 16 m not to be significantly influenced by each other.

This distance affects directly the design of the trenching patterns.

TRENCHING PATTERNS

The design of trenching patterns is governed by two rules. First the ice cover has to be cut in pieces of small areas in order to facilitate their eventual evacuation at break up. The second rule is the ease of realisation, taking into account security (influence zone) and the mode of operation of the trencher (not able to make curved patterns).

Five differents tests patterns were choosen. The patterns are shown on Figure 2. In this figure arrows are indicating the direction of movement of the trencher.

REALISATION OF TRENCHES

The trenching was done at a time where there was 50% chance of break up.

a) Choice of trencher

Many companies have already some trenchers for ground work. These trenchers are usually operationnal in ice without any modification.

The choice of the trencher is then governed by the bearing capacity of the ice cover. A Case "DH4" trencher was choosen. This tractor is weighing about 2500 kg and has a chain width of 150 mm (Figure 3).

b) Tracing of trenches

The patterns were traced with the help of orange colored spray paint. Taking into account the eventual formation of shore cracks, the patterns of trenches were limited to the central part of the river.

c) Realization

Two access roads were available for the trencher.

Each of the first four patterns were realized in about 20 minutes. The advancing rate of the trencher was then a little less than 4 m/minute. For the fifth pattern, which is rectilinear, the advancing rate was about 8 m/minute.

The trenching of all the 600 m long meander took 8 hours, or a normal working day.

The trencher had no difficulty to move on the ice cover with local snow accumulation up to 30 cm, but usually little snow on it.

d) Security

Great attention was given to security when working on ice.

The tractor operator was wearing a life-jacket and was followed by a person with a lifebuoy tied to a rope.

A continuous survey of ice thicknesses and of the dangerous spots was done as the work was proceeding.

BREAK UP

The break up of the ice cover happened almost a few days after trenching of the patterns. But, then, a fall in air temperatures delayed it one month later.

Fortunately, at the time of break up observations were made from an helicopter so that a film was taken showing the ice cover just before, during and after break up of the ice in this river meander.

The river discharge was then 130 m^3/s. The ice cover was completely separed from the river banks and the 150 mm wide trenches were then up to 300 at 600 mm wide. The trenches had also generated several supplementary cracks in the ice cover.

At break up the ice cover broke in pieces whose forms followed those of the patterns. An uncut little meander, in the lower part of the river prevented the ice pieces to be freely evacuated (Figure 4).

All the patterns reached their goal. Finally it was found that the first pattern permitted to break the ice cover in the smallest pieces.

CUTTING TRENCHES VS BLASTING

A comparison of this method with the blasting experience held in 1982, showed that it has many advantages. They are:

- trenching breaks more ice than blasting;
- is a third as costly;
- has no environmental effects;
- do not required external security devices during execution.

The disadvantages are principally that the cutting of trenches could only be used when the bearing capacity of ice is adequate to support the loads. This may be very critical just prior to break up.

CONCLUSION

The experience of cutting trenches in the ice cover to prevent ice jams attained completely its goals.

This method, whose major advantage is that it does not have any environmental effect, should be used more often in the future to prevent flooding in rivers by ice jams.

REFERENCES

BOLSENGA, S.J., (1968), River ice jams a litterature review. Department of the army, Lake survey district, Corps of Engineers, United States, 568 p.

DEUGO, D., (1973), Ice control on the Rideau river, Ottawa. Seminar on Ice Jams in Canada, Alberta, Technical memorandum No. 107, p. 155-157.

MICHEL, B., (1971), Winter regime of lakes and rivers. U.S. Army Corps of Engineers, Monograph III-B1a, 130 p.

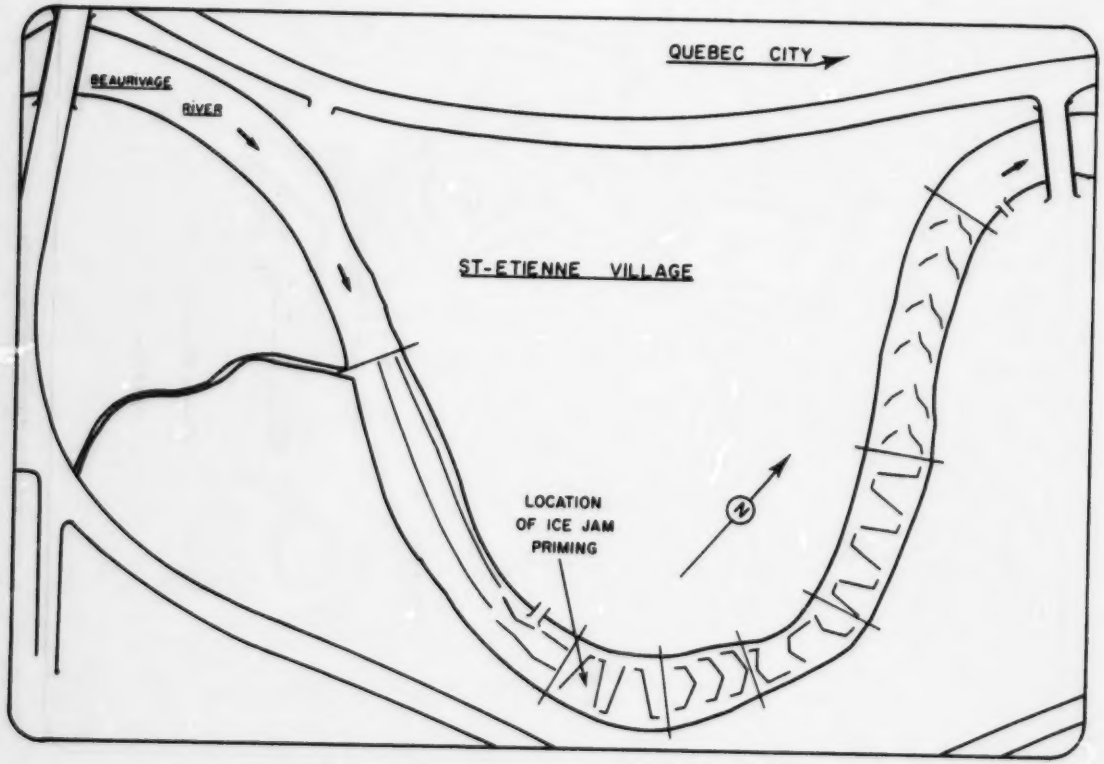


Figure 1 - General map of the river and the site

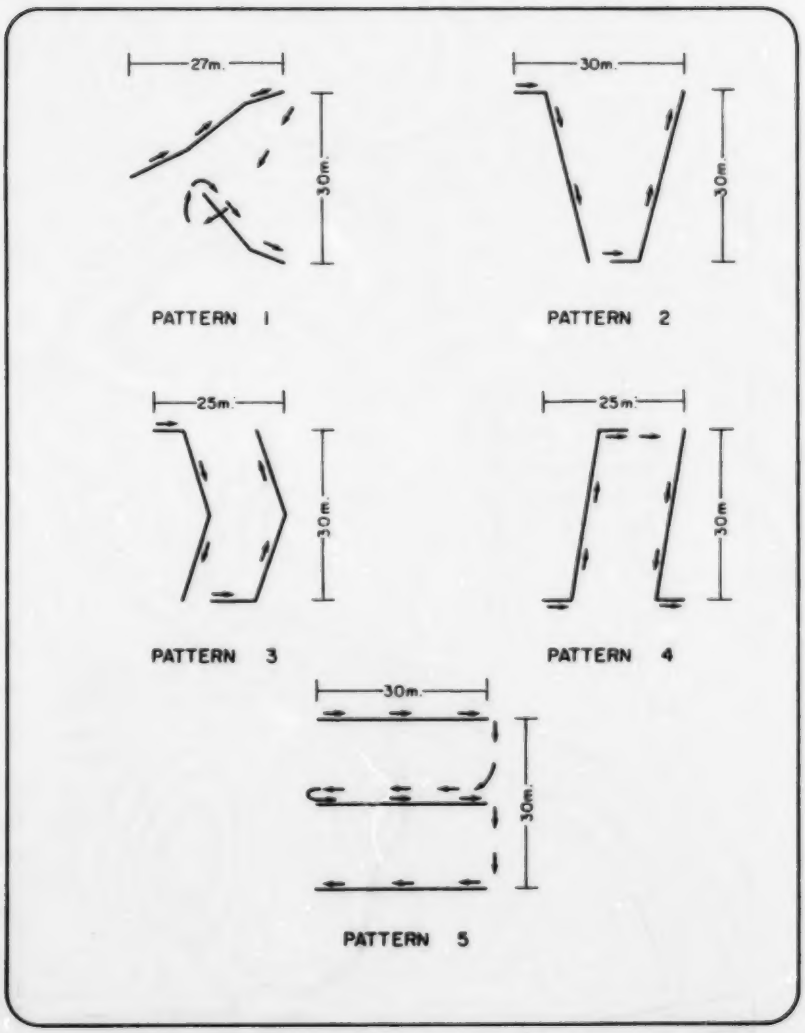
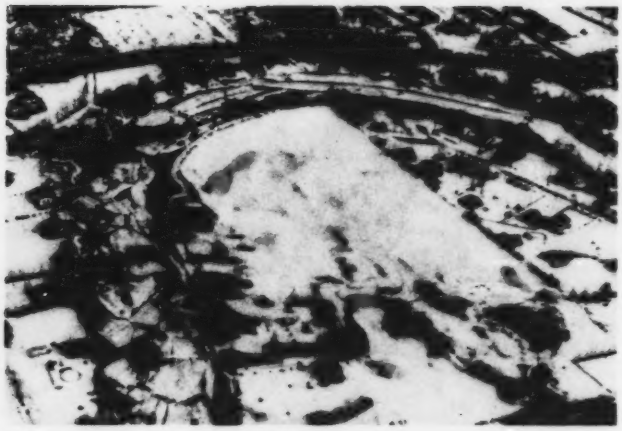


Figure 2 - Pattern of ice cutting

Figure 3 The "DH4" trencher in operation in ice

(before)



(after)

Figure 4 - Pictures of break-up
of the river reach

DISCUSSION

D. Smith

Was the possibility of children playing on ice and falling through considered?

Reply

Yes, but the trenches are only six inches wide and nobody can fall through such a narrow opening. Furthermore, signs are posted all along the shore to indicate that the ice is dangerous.

S. Ismail

It was mentioned that one pattern of trenching produced more cracks than the others. Is that due to the mechanical properties of the ice cover or to the change in the rate of melting caused by radiations absorption?

R. Abdelnour

During your presentation you have mentioned that five ice cutting patterns were tried. Could you please give your opinion, based on your observations of the results, which pattern was most effective to initiate secondary cracks and facilitate the flow of ice downstream?

Reply

The best pattern of trenching was found to be the one which initiated more secondary cracks as suggested by Mr. Abdelnour. The experiment has shown that the alternate crossed trenches at about 45° to the shore were the best to do just that. Some analysis or model tests would be required to confirm that the stresses caused by the water flow would then fail more easily the ice with this pattern than others. The effect of melting by radiations was not considered important.

D. Foulds

Did the ice have free discharge downstream when breakup began?

Reply

Yes, just downstream from the lower river curve. This is required if the operation has to be successful. Otherwise, the ice would just pile up a little more downstream.

S. Dumont

Is there any optimal depth for the trenches or did you go just the length of the saw?

Reply

The idea is to cut right across the total ice cover thickness. Because the ice is never more than 60 cm to 75 cm thick at that time, the standard cutters on trenchers are long enough to do it.

B. Pratte

I noticed in your photographs of the DH4 trencher that the snow had not been removed ahead of the trencher, allowing you to see presence of cracks before the machine arrives. Could you elaborate on how the field crew preceding the DH4 assessed the bearing capacity and safety of the ice cover. Could the trencher itself not have a snowblower and plow and attached or preceding it? If you had not taken ice block samples from the planned path of the DH4 (before the trenching) and analysed for S_2 ice thickness to get bearing strength, what factor of safety or reduction factor would you apply to the estimated bearing capacity?

Reply

At the time of trenching a good proportion of the snow cover had already melted so wet cracks (on shore for example) could easily be seen either because they were water surface free or because the infiltration had come to the top of the snow layer.

If the operation had been done sooner in the winter with a thick snow cover it might not have been so easy to see these cracks and some snow clearing might have been needed.

A coefficient of safety on only about 1.2 was used

to compute the bearing capacity of the cover, taking into account the detailed analysis of the ice cover characteristics. If same spot checks had been made of the thickness only, we would not have taken a safety factor of lower than 2.0.

U. S. Panu

Would the effectiveness of one or the other pattern in the prevention of an ice jam be influenced by the sequence in which they were evaluated in the paper? By examining only one case study, is it possible to conclude which pattern should be preferred over the other?

Reply

I do not think the order of the arrangement of patterns has much to do with their effectiveness. In each case a jam is pushing the broken ice at its downstream end, whatever be the order used. There is little doubt that more experiments would be very useful to conclude more positively on the best pattern to use.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

DOCUMENTATION AND ANALYSIS OF THE WATER LEVEL PROFILE THROUGH AN ICE JAM, MACKENZIE RIVER, N.W.T.

G. Rivard[1], T. Kemp[2] and R. Gerard[3].

ABSTRACT

In the spring of 1983, the water level profile through a long ice jam upstream of Norman Wells on the Mackenzie River, N.W.T., was determined in the field using a simple approximate technique developed for such inaccessible locations. An attempt was then made to simulate this profile, using gradually varied flow calculations, to determine estimates of the ice accumulation thickness and roughness variation. The field documentation and results of the preliminary analysis are described.

[1] Graduate Student, Department of Civil Engineering, U. of Alberta, Edmonton, Alberta.
[2] Research Engineer, Esso Resources Ltd., Calgary, Alberta.
[3] Professor, Department of Civil Engineering, U. of Alberta, Edmonton, Alberta.

RESUME

Au printemps de 1983, le profil de la ligne d'eau le long d'un embâcle de glace en amont de Norman Wells sur la rivière Mackenzie, T.N.O., a été mesuré en nature utilisant une simple technique approximative développée pour de tels inaccesibles endroits. Le profil obtenu a ensuite été simulé, en utilisant un calcul de courbe de remous, pour estimer l'épaisseur et la rugosité de l'embâcle de glace. La collection des données en nature et les résultats de l'analyse sont décrits.

INTRODUCTION

Although methods to analyse ice jams have been available for almost twenty years, application of these methods is still uncertain due to the unknowns involved. In particular, the thickness and roughness of the accumulation have to be evaluated by indirect methods as no safe techniques have been developed to measure directly either the thickness of the ice jam or the velocity profile under the jam. Yet, there is a dire need to incorporate consideration of ice jams into flood plain studies and various other design situations in northern regions so that even indirect estimates of these parameters are of value.

This study discusses the field documentation of an ice jam on the Mackenzie River in the Norman Wells area and describes the calibration and use of the backwater hydraulic model HEC-II, with its new ice option, to simulate the measured ice jam water level profile and hence determine the combination of thickness and roughness that gave the best fit to the measured water profile.

GEOMORPHIC CHARACTERISTICS AND BREAKUP PROCESSES

The Mackenzie River is a large river in the Northwest Territories that flows from Great Slave Lake approximately 1600 km northwestward to the Beaufort Sea (Figure 1). A plan of the reach of interest is given in Figure 2. The low water longitudinal profile of the river in the study area is shown in Figure 3. It was obtained from the low water datum established in 1973 by Public Works of Canada (Mackenzie River Investigation, Final reports, 1976). Also shown is the open water profile measured between August 17 and 19, 1983, as part of this investigation, when the discharge was 10 800 m^3/s.

The breakup processes and ice jam locations along the Mackenzie River between Fort Simpson and Norman Wells have been documented previously (see, for example, Mackay and Mackay, 1973; Kamphuis and Moir, 1983). Ice jam formation and the surges produced by their release have been found to be a very significant factor in breakup progression and in the formation of other ice jams further downstream (Parkinson, 1982; Stagg et al., 1983). In the reach of interest, two locations seem to be particularly prone to ice jam formation. The first is at Fort Norman where, despite the large width, shoals off the Great Bear River confluence and Windy Island often cause ice jamming. The other is downstream of Norman Wells (at km 928), where the width of the channel is narrow.

ICE JAM DOCUMENTATION, 1983

On April 22, 1983, the ice thickness at Norman Wells was measured by WSC to be 1.7 m. On May 22, at approximately 22:30, the first ice movement was reported in the Fort Norman area (km 828).

A flight was taken from Norman Wells to Fort Norman early on May 23 to observe the ice conditions at this location. On arrival in Fort Norman at about 9:45, the jam that had formed in front of the settlement was stable and no ice movement could be noticed from the bank. The ice jam was keyed just downstream of Fort Norman (at km 830) and extended upstream for about 70

km with its large floe - mush ice interface at km 816. Ground observations of the ice along the bank indicated the ice was in fair condition, with little candling apparent. At about 11:15, small movements of the ice near the shoreline at Fort Norman were noticed and, shortly after, the entire ice jam began to move. The velocity of the flow caused by the ice jam release was estimated to be over 5 m/s.

At 12:35 the ice sheet in the Norman Wells area was still in fairly good condition with no noticeable cracks up to Halfway Island (km 856). However, numerous large open water areas could be seen around the islands. The ice sheet by Gaudet Island (km 845) was cracked at several locations and at the Little Bear River confluence (km 842) there was an open reach of about 1 km.

At 13:20, the surge produced by the ice jam movement at Fort Norman triggered the first noticeable ice movement at the Off-Site Dock, about 4 km upstream of Norman Wells. Major ice movement at the Government Dock in Norman Wells began at 15:07. At about 15:40, ice movement ceased. The ice sheet in front of Norman Wells had moved about 0.5 km and was left fractured into large floes. The water level variation at Norman Wells during this event is shown in Figure 4. The resulting jam appearance is shown in Figure 5 and the complete ice jam plan in Figure 6b.

Later in the evening (20:30), when water levels in Norman Wells were slowly decreasing, another flight was conducted to record photographically the water profile along the ice accumulation using the procedure described by Gerard (1975). Photos were first taken from high level to provide views of the toe, head and general features of the jammed reach and then at low level against the river banks to document water levels at noted locations.

Later in the year, on August 17, these photographed water levels were tied in to W.S.C. bench marks available along the reach to obtain the complete water profile shown in Figure 6a. Due to the crudeness of the measurement method, errors up to 0.8 m are likely possible. However, at the time the water profile was being documented, reliable water level measurements were obtained in Norman Wells and in Fort Norman. These accurate measurements are identified as such in Figure 6a, along with other reliable measurements obtained from the photos where excellent reference points were available.

ANALYSIS

Model Calibration

The HEC-II model was first calibrated using the open water profile measured during the August survey. Based on the rating curve for Norman Wells provided by W.S.C., the discharge at this time was 10 800 m^3/s. Cross-section data were obtained from the hydrographic charts (PWC, Mackenzie River Investigation, 1976). Examples are shown in Figure 7. A total of 26 cross-sections were used to describe the 100 km reach. The calculated profile is shown in Figure 2. The values of "n" were varied to reflect expected changes in roughness based on available localized river bed data (Figure 2). In general, low values of "n" were required to get reasonable agreement with the measured profile. This seems justified by the nature of the river bed as revealed by the field boring data given in Figure 2.

Simulation of measured ice jam profile

The HEC II model has been recently modified to take account of the presence of an ice cover in backwater calculations (Calkins et al., 1982). This model was used to analyse the water profile caused by the ice jam. The ice cover thickness and the estimated value of "n" for the ice cover are input at each cross-section. The program utilizes the Belokon-Sabaneev formula to determine a composite "n" value for the channel.

A fundamental requirement is knowledge of the discharge while the ice jam was in place: this is very difficult to obtain. Most of the time the usual WSC gauges malfunction or give a poor measurement at break-up. In the present case, the gauge located in Norman Wells was effectively put out of order at about 14:30 on May 23, 45 minutes before the first noticeable ice movement.

The mean daily discharge at Norman Wells as estimated by WSC was 22 700 m^3/s on the day preceding the jam event. This was based on an interpolation of the discharges recorded at Fort Simpson (Km 338) and Arctic Red River (Km 1430). Other independant estimates are possible. Using the water level recorded photographically downstream of the ice jam in the intact ice sheet region, the discharge was computed using the cross-section data and assumed bed and ice cover roughnesses. This gave about 10 000 m^3/s. A second estimate utilized the cross-section at Fort Norman (km 828) and the more reliable water level recorded there. At the time, this reach was in an almost open water condition, with only some ice debris still flowing down toward the head of the jam. The main concern was whether or not the location was in the backwater region of the jam. This has not yet been ascertained. Assuming it was not affected by backwater, the discharge was estimated to be 14 700 m^3/s. If it was in the backwater the estimate would be lower and closer to the previous estimate. A value of 14 700 m^3/s was accepted for use in these preliminary calculations.

The ice jam reach could be divided into four distinct sub-reaches with respect to the ice conditions. Downstream of approximately km 915, the ice sheet could be considered to be intact except for some large cracks. The next sub-reach, from km 915 to approximately km 870, was composed of large floes mixed with slush at the downstream end, as shown in Figure 8, and what seemed , from the air, to be relatively compact slush ice. From there to about Km 856, the slush accumulation appeared somewhat less compact. In the last reach, up to Fort Norman (Km 828), ice debris were still flowing toward the ice accumulation.

Based on the above subdivision, roughnesses and thicknesses were estimated using the backwater model to simulate the water profile. The ice thickness for the intact sheet was taken as 1.7 m with a roughness of 0.020. In sub-reach 2, the toe and main body of the jam, the best fit roughness was estimated for assumed thicknesses between 3 and 5 m. The results of the analysis are given in Table 1. Both Manning's n and the hydraulic roughness "k_i" for the ice are given. The major advantage of using the hydraulic roughness k_i instead of "n" is that the former can be directly and easily related to the size of the boundary roughness elements. Experience suggests that $\Delta < k < 3\Delta$, where Δ is the equivalent diameter of the larger roughness projections.

Based on the ice accumulation appearance, shown in Figure 8, the best choice seemed to be n_i= 0.045 (k_i= 3.3 m) and t = 4 m. With these values, a good fit to the measured profile was maintained until km 870. Upstream of that point a thickness of 1.9 m, with the same roughness, gave a better fit. In

the last sub-reach (approximately km 856 to km 828), the resistance of the ice cover was ignored as the ice was still flowing toward the head of the jam at the time of the measurement.

Estimation of accumulation internal friction parameter

Based on the results of the above analysis, an estimate of the internal friction parameter can be obtained from equilibrium thickness calculations (Pariset et al., 1966). The equation of stability of a simple wide jam is given by:

$$\mu \rho_i (1 - s_i) g t^2 - \left[(g \rho_i S - 2C_i/B) B \right] t - \tau_i B = 0 \qquad (1)$$

where μ is the internal friction parameter, ρ_i is the density of the ice, s_i is the specific gravity of the ice, g is the acceleration of gravity, t is the thickness of the accumulation, S is the slope of the water surface, B is the channel width at the bottom of the accumulation, τ_i is the shear stress on the bottom of the accumulation and C_i is a factor for cohesion within the accumulation

The hydraulic radius R and the mean depth under the jam are given by:

$$R = A/2B = H/2 \qquad (2)$$

where A is the area of the waterway under the ice. Assuming that the total average velocity and the average velocities in the flow areas associated with the ice cover and the bed are equal, the depth of flow associated with the ice cover can be shown to be:

$$H_i = H / (1 + (k_r)^{-0.25}) \qquad \text{with } k_r = k_i/k_b \qquad (3)$$

where k_i is the ice cover roughness and k_b the bed roughness (taken as 0.05 m from open water computations). The shear stress on the underside of the ice cover can then be determined by:

$$\tau_i = g \rho H_i S \qquad (4)$$

Using the parameters determined in the previous section, equation (1) can be used to estimate the internal friction parameter. This was done for 3 cross-sections in the reach between km 882 to 870 where the slope of the ice jam seemed to be parallel to the open water profile and where the channel was undivided (see Table 2 for a summary). Computed values of the friction parameter varied between 1.6 and 2.0: this is within the range of 1.0 to 2.0 reported by other investigators.

CONCLUSIONS

In 1983 the water profile through an ice jam formed on the Mackenzie River upstream of Norman Wells was documented. The profile was later simulated using the HEC-II gradually varied flow program calibrated to open water conditions, with the thickness and roughness of the ice accumulation along the jam being varied to get the best agreement with the measured profile. The values of the ice cover roughness were selected based on the surface appearance of the ice cover. Although the jam formed under quite unsteady circumstances, it was found in these preliminary calculations that good agreement could be obtained along the whole jam without appeal to large thickness or grounding in the toe region. Furthermore the thickness found necessary in the body of the jam agreed reasonably with that given by the assumption of an equilibrium section and uniform flow with values of the accumulation internal friction parameter varying from 1.6 to 2.0.

It seems therefore that, even in what seems a quite complex circumstance, ice jam profiles can be calculated using simple gradually varied flow calculations, reasonable assumptions about hydraulic roughness and simple estimates of ice jam thickness.

ACKNOWLEDGEMENTS

This project was funded by Esso Resources Canada Limited and permission to present this paper is gratefully acknowledged. Supplementary support was provided by the National Research Council of Canada. Special thanks are extended to Messrs. P. Stagg, Esso Resources, and J. Hajash, RCMP officer in Fort Norman, who both assisted during the field documentation.

REFERENCES

Calkins, D.J., Hayes, R., Daly, S.F., and Montalvo, A., (1982). *"Applications of for Ice-covered Waterways,"* Journal of the Technical Councils of ASCE, Vol. 108, No. TC2, November, pp. 241-248.

Gerard, R., (1975). *"Preliminary Observations of Spring Ice Jams in Alberta"*, Proc., 3rd International Symposium on Ice Problems (IAHR), Hanover, New Hampshire.

Kamphuis, J.W., Moir, J.R., (1983). *"Ice Breakup and Jamming Observations along the Mackenzie River"*, Canadian Journal of Civil Engineering, Vol. 10, No. 1.

Mackay, D.K. and Mackay, J.R., (1973). *"Breakup and Ice Jamming on the Mackenzie River, N.W.T."*, Report on Hydraulic Aspects of Northern Development, Glaciology Division, Water Resources Branch, Environment Canada, Ottawa.

Mackay, D.K. and Mackay, J.R., (1973). *"Location of Spring Ice Jamming on the Mackenzie River, N.W.T."*, Technical Report No. 8, Glaciology Division, Water Resources Branch, Environment Canada, Ottawa.

Pariset, E., Hausser, R. and Gagnon, A., (1966). *"Formation of Ice Covers and Ice Jams in Rivers"*, Journal of the Hydraulics Division, ASCE, pp. 1-24.

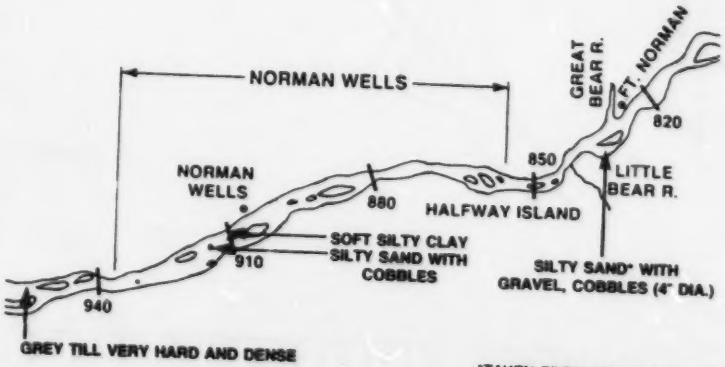
Parkinson, F.E., (1982). *"Water Temperature Observations during Breakup on the Liard-Mackenzie River System"*, Proceedings of the Workshop on Hydraulics of Ice-Covered Rivers, June 1-2, Edmonton, Alberta, pp. 252-265.

Public Works Canada (1976). *"Mackenzie River Investigation - Proposed Navigation Channel Improvements"*, Final Report, Volumes 2, 3, 4. Public Works Canada, Western Region.

Stagg, P.W., Kamp, T.S., Moir, J.R., (1983). *"Analysis of Peak Water Levels in the Mackenzie River at Breakup"*, Symposium on Science and Arctic Hydrocarbon Exploration: the Beaufort Experience, Sept. 29-30, Whitehorse, Yukon.

U.S. Army Corps of Engineers, (1976). Hydrologic Engineering Center, *"HEC-II Water Surface Profiles, Generalized Computer Program, Users and Programmer manuals"*, Davis, Ca., November. (Updated Version 1981 - Modification 55 for Ice Cover Analysis).

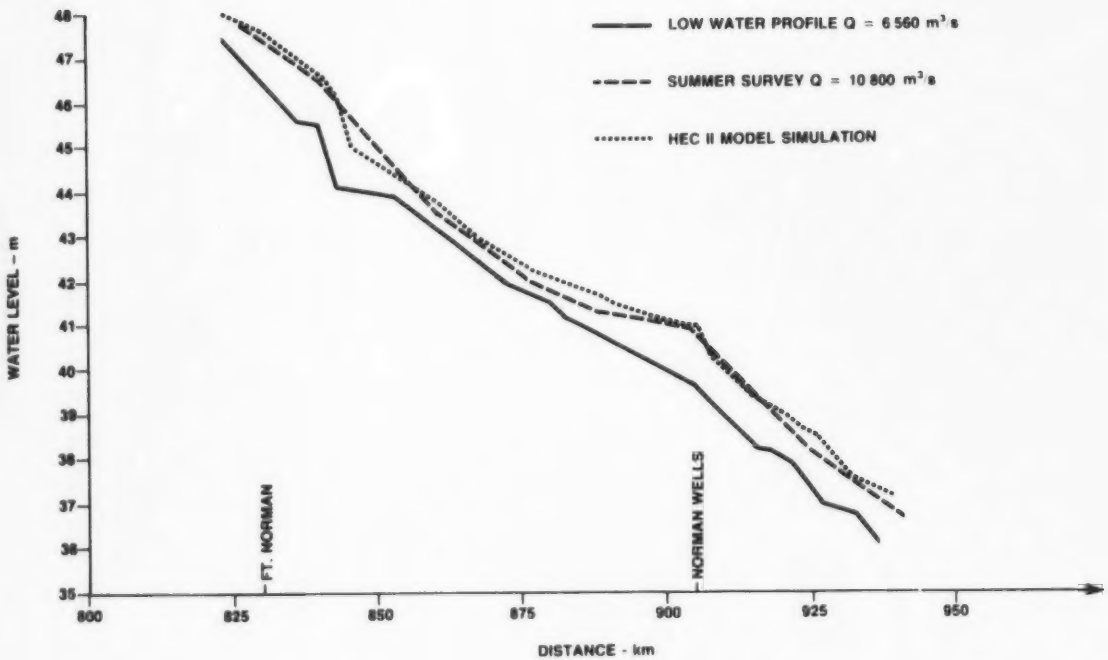
Figure 1 LOCATION PLAN FOR THE STUDY AREA



*TAKEN FROM MACKENZIE RIVER INVESTIGATION REPORTS 1976

Figure 2 PLAN OF THE STUDY REACH

Figure 3 LONGITUDINAL PROFILE AND MODEL CALIBRATION FOR OPEN WATER CONDITIONS IN REACH OF INTEREST



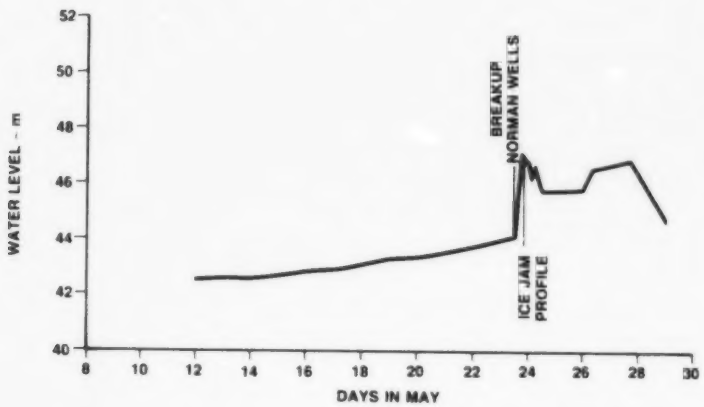


Figure 4 WATER LEVELS AT NORMAN WELLS DURING THE 1983 BREAKUP PERIOD

Figure 5 GENERAL VIEW OF ICE JAM LOOKING DOWNSTREAM TOWARD NORMAN WELLS

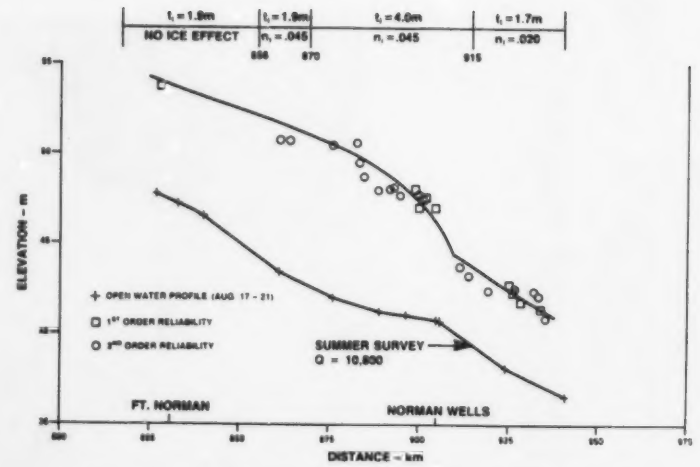


Figure 6a MEASURED AND COMPUTED ICE JAM PROFILE

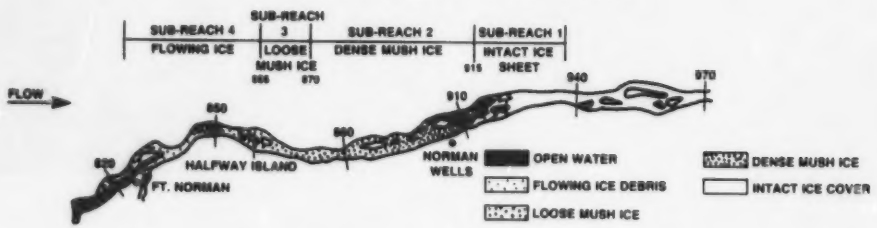


Figure 6b PLAN OF 1983 ICE JAM

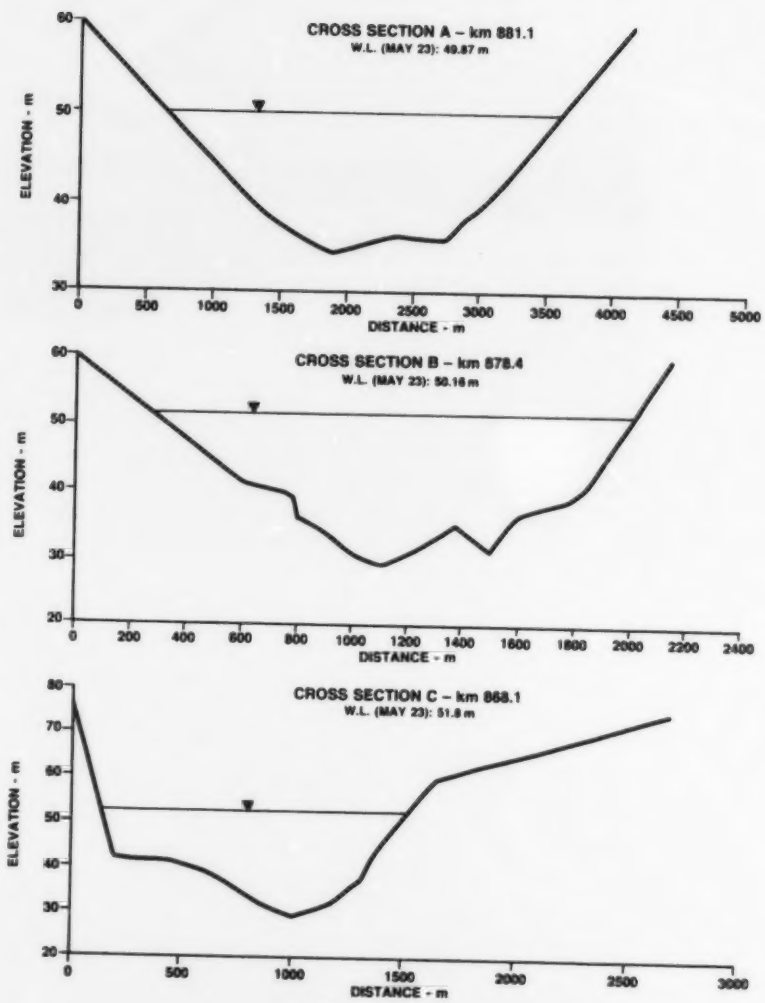


Figure 7 MACKENZIE RIVER CROSS SECTIONS WITHIN THE EQUILIBRIUM PORTION OF THE 1983 ICE JAM

Figure 8 SURFACE CONFIGURATION OF THE ICE JAM TOE IN THE NORMAN WELLS AREA

TABLE I SUMMARY OF COMPUTER SIMULATIONS
(CROSS SECTION A – km 881.1)

t, (m)	N_i*	K_c** (m)	K_i*** (m)
3.0	.062	2.4	16.5
3.5	.055	1.35	8.0
4.0	0.45	0.68	3.3
4.5	.035	0.26	0.85
5.0	.028	0.11	0.22

* CALCULATED SO THAT THE CURVE FIT THE PROFILE IN SUB-REACH 2 (TOE AND MAIN BODY OF THE JAM)

** CALCULATED FROM COMPUTER PRINTOUT AND USING THE STANDARD SEMI-LOGARITHMIC EXPRESSION:

$$\frac{V}{V_*} = 2.5 \ln \frac{R}{K_c} + 6.2$$

*** CALCULATED USING COMPUTED K_c, AN AVERAGE K_b OF 0.05m AND THE RELATION

$$K_c = K_i \left[\frac{1 + (\frac{K_i}{K_b})^{-0.25}}{2} \right]^4$$

WHICH IS THE BELOKON-SABANEEN EQUATION REWRITTEN WITH HYDRAULIC ROUGHNESSES INSTEAD OF MANNING'S "n"

TABLE II SUMMARY OF CALCULATED ICE JAM CHARACTERISTICS
(Q = 14 700 m^3/s)

CROSS SECTION LOCATION (km)	881.1	878.4
S (m/m)	.000111	.000119
B (m)	2164	1331
H (m)	15.07	21.26
A (m^2)	19366	15431
V (m/s)	0.76	0.95
t (m)	4.0	4.0
K_s (m)	0.68	0.67
K_b (m)	0.05	0.05
K_i (m)	3.3	3.2
T_i (Pa)	7.21	10.0
μ	2.0	1.6

DISCUSSION

John Anderson

1. In the past I have been asked about the likely impact on ice jam formation of the ice road generally built across the Mackenzie River at Norman Wells. What is your feeling about this?

2. The more advanced progression of the 'mush' ice along the left side of Bear and Goose Islands is likely reasonable as the thalweg lies along that side of the islands.

Reply

As mentioned in the paper ice jams generally 'key' on the ice in the narrow reach at about km 928, well downstream of Norman Wells. Given this, and the very small extent of the ice bridge compared to the massive size of the river at Norman Wells, it is unlikely the ice road at this site would have any significant influence on the ice jam problems in this reach.

S. Ismail

1. As there were changes in water level during the observation period, the flow was unsteady. How do you then justify the use of HEC-2 for this situation?

2. What modification did you make to HEC-2 to make it capable of handling quasi-steady flow?

Reply

The writers appreciate the questions by Dr. Ismail as they permit a clarification of the premises underlying the analysis.

1. As pointed out, the flow was unsteady. However, it is evident from Figure 4 that there were degrees of unsteadiness. When the first ice movement occurred there was a very dynamic change in the water levels (≈ 0.48 m/h). Nevertheless, when the water profile was measured 5 hours later the water levels were decreasing at a much slower rate (≈ 0.07 m/h). It was therefore felt reasonable

to treat this as a 'quasi-steady' situation - i.e. one for which an assumed steady state is a close approximation.

2. As explained above, none was necessary.

F. Parkinson

A point of clarification: there is a change in slope at Norman Wells (≈ km 905) under open water conditions. The sudden change in the ice-covered profile occurred at the jam key just downstream of Rader Island, at about km 925.

ERRATA

In Table 2 the following changes should be made to line 3:

H to h_j: 15.07 to 8.95; 21.26 to 11.6

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21,1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

CONFERENCE PRESENTEE AU TROISIEME ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU, FREDERICTION, 20 JUIN 1984

PROTECTION CONTRE LES INONDATIONS EN PRESENCE DES GLACES

RIVIERE SAINT-CHARLES A QUEBEC

Jean Monfet[1] et Albert-Réal Tremblay[1]

RESUME

Des inondations en présence des glaces se produisent dans un quartier urbanisé de la ville de Québec, le long de la rivière Saint-Charles. Une zone d'écoulement rapide en amont génère de la glace qui s'accumule dans le tronçon aval à écoulement plus lent. Lors de débâcles hâtives, des glaces du tronçon plus rapide se décrochent, s'appuient sur le couvert solide aval et créent des rehaussements d'eau importants et des inondations. Le ministère de l'Environnement a étudié ce cas et proposé l'implantation d'un seuil et d'une estacade fixe en amont du secteur urbanisé à protéger afin de solutionner ce problème. Le but de l'ouvrage est de retenir les glaces brisées venant de l'amont lors de l'augmentation du débit. Des objectifs secondaires sont aussi atteints; la présence du seuil en diminuant les vitesses, hâte la formation du couvert et restreint la surface génératrice de frasil. Un certain volume d'emmagasinement pour le frasil est aussi créé. Cet ouvrage a été construit à l'été 1983 et les observations de l'hiver confirment son utilité.

[1]Ingénieurs, Direction des Ouvrages hydrauliques, ministère de l'Environnement du Québec, Québec.

FLOOD PROTECTION WITH ICE COVER ON "THE RIVIERE SAINT-CHARLES" QUEBEC CITY

ABSTRACT

Floods caused when ice cover still there occur in an urban part of the Quebec city along the "riviere Saint-Charles". Ice produced in a reach of high velocities gets to a reach of low velocities where it accumulates. At winter break-up, solid ice cover of the low velocities reach blocks the incoming ice floes. As a consequence, water level rises and floods occur. The Ministry of the Environment of Quebec studied this case and proposed the installation of a weir and a fixed boom on the reach, upstream of the urban part, in order to solve this problem. The weir and the boom have to retain the ice coming from upstream when the flow discharge is rising. The weir also permits to attain others purposes: it reduces the flow velocity in the upstream reach such as an ice cover is early formed and then area where the frazil is generated is reduced; besides, the weir builds up storage room for some frazil. The weir and the boom were build in the summer 1983 and the observations of the last winter tend to prove their usefulness.

INTRODUCTION

La ville de Québec n'est pas réputée pour avoir des inondations étendues dans le temps ou dans l'espace. Les débordements d'une rivière comme la Saint-Charles ou la du Berger sont, et de beaucoup, moins spectaculaires que les crues de rivières telles la Chaudière, le Haut-Richelieu ou l'Outaouais. Il n'en demeure pas moins qu'une inondation même peu étendue sur le territoire de la ville de Québec peut avoir des conséquences catastrophiques. En effet, la densité de l'habitat pouvant être inondé est élevée, impliquant ainsi des dommages importants (Bouillon, 1981).

Les rives des rivières Saint-Charles et du Berger sont parfois l'occasion d'inondations importantes en présence des glaces et pour solutionner ce problème, le ministère de l'Environnement du Québec a réalisé à l'été 1983 la construction d'ouvrages permettant la rétention des glaces de ces rivières en amont des quartiers urbanisés à protéger. Les inondations à l'eau libre des rives urbanisées des rivières Saint-Charles et du Berger sont peu importantes. Une crue centennale produit des débordements latéraux peu étendus dans les secteurs urbains.

Le présent travail décrit les différentes étapes qui ont conduit à la mise en place de l'ouvrage de rétention des glaces sur la rivière Saint-Charles à Québec. Cet ouvrage consiste en un seuil surmonté d'une estacade fixe. Après avoir présenté la rivière Saint-Charles et décrit la problématique des inondations le long de cette rivière, nous nous attarderons au secteur souffrant de façon plus aigüe des inondations en présence des glaces. Une analyse hydraulique de la rivière sera faite plus en détail pour ce tronçon et le comportement des glaces sera décrit tout comme les dommages occasionnés. La solution retenue pour régler ce problème sera présentée ainsi que les résultats obtenus au cours de la première année d'exploitation de l'ouvrage.

PROBLEMATIQUE

Présentation de la rivière

La rivière Saint-Charles est située au nord du fleuve Saint-Laurent et s'y déverse à la hauteur de la ville de Québec, plus précisément à l'est du cap Diamant. La superficie de son bassin versant à l'embouchure est de 512 kilomètres carrés (figure 1). La rivière Saint-Charles prend sa source dans le lac du même nom situé à environ 15 kilomètres au nord de la ville de Québec. Après un parcours lent et sinueux sur quelques kilomètres, soudain l'écoulement s'accélère, dévale les chutes de Loretteville et continue à grande vitesse jusqu'au pied du premier contrefort des Laurentides. De là jusqu'au fleuve, l'écoulement redevient calme. C'est dans ce tronçon inférieur que se situe la zone urbaine à forte concentration. En amont du lac Saint-Charles, le bassin versant est montagneux et très faiblement urbanisé.

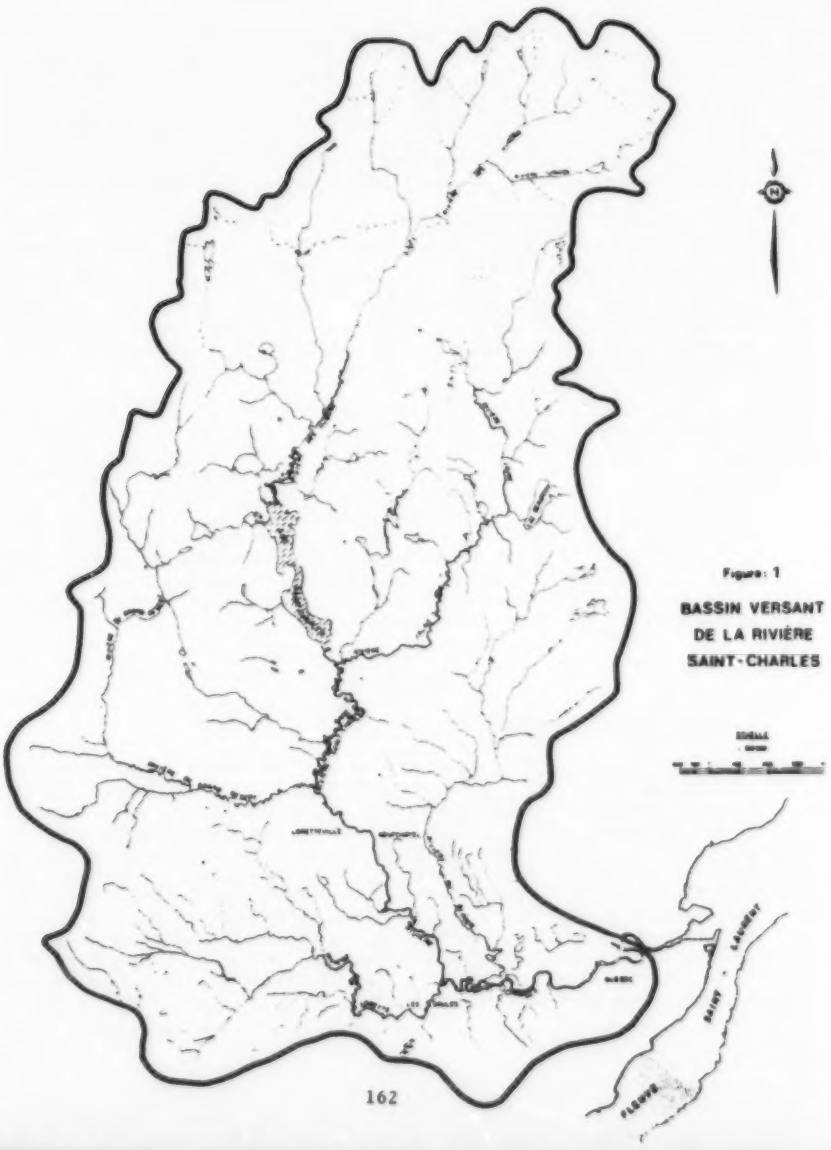


Figure: 1

BASSIN VERSANT DE LA RIVIÈRE SAINT-CHARLES

La rivière Saint-Charles est utilisée à de multiples fins. Un seuil aménagé près de son embouchure crée un plan d'eau en plein centre-ville de Québec, ce qui a permis l'aménagement d'un parc urbain linéaire. Ce même plan d'eau est utilisé comme patinoire en hiver. Cette dernière, aménagée sur 3 kilomètres environ, soit la presque totalité de la longueur de la retenue, est très fréquentée, particulièrement lors du Carnaval de Québec. La rivière Saint-Charles constitue aussi la principale source d'approvisionnement en eau potable de la ville de Québec. Un barrage aménagé à la sortie de ce lac permet d'utiliser le lac à des fins d'emmagasinement pour assurer l'approvisionnement en eau de la ville. Ce lac constitue un plan d'eau de 3,5 km^2 utilisé à des fins récréatives, entre autres pour la voile légère. La pêche est aussi pratiquée sur les différents affluents de la rivière, ainsi que sur le tronçon principal. Un programme d'ensemencement de la truite a en effet permis de réintroduire cette espèce sportive dans la partie supérieure de la rivière.

Les inondations

Cette rivière, en plus des aspects positifs mentionnés précédemment, occasionne malheureusement quelques inconvénients, dont celui des inondations. Ces dernières affectent principalement deux tronçons en aval du lac Saint-Charles et deux phénomènes distincts en sont la cause, soit les inondations à l'eau libre et les inondations en présence de glace.

Une vue en profil de toute cette portion assujettie aux inondations permet de mieux identifier les zones inondables (figure 2). La rivière peut être divisée en cinq tronçons différents:

- le tronçon du bassin Saint-Charles
- le tronçon de Duberger
- le tronçon de Neufchatel - les Saules
- le tronçon de Loretteville
- le tronçon de Château d'Eau

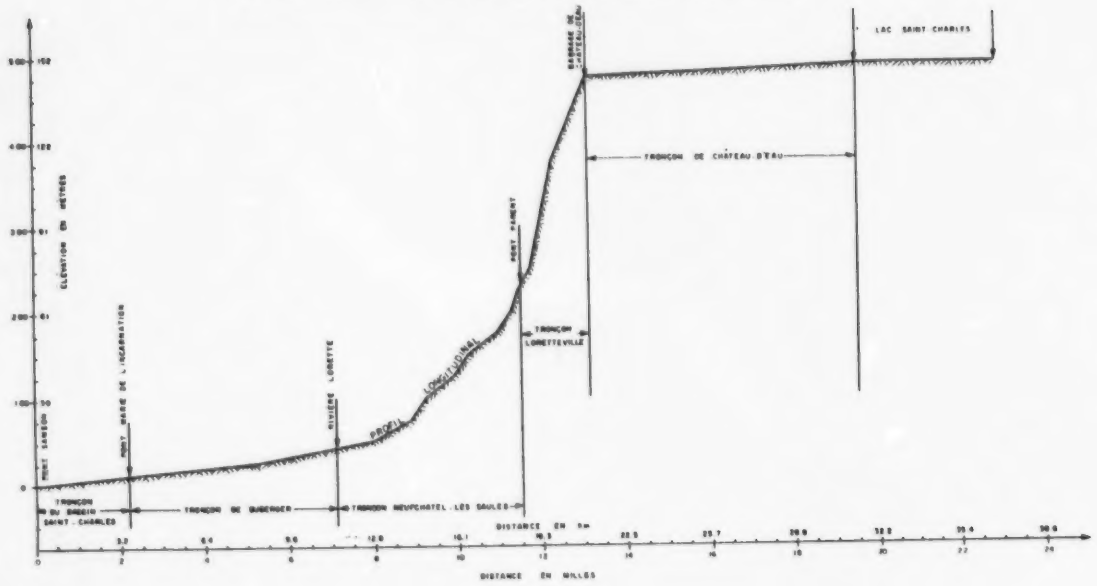
Le tronçon du bassin Saint-Charles

Le bassin Saint-Charles s'étend de l'embouchure de la rivière au pont Marie de l'Incarnation et est créé par le refoulement des eaux du barrage Samson érigé près de l'embouchure. Les vitesses d'écoulement pour des débits normaux d'hiver, soit lors de la formation du couvert de glace, sont très faibles (0,1 m/s), si bien que le couvert se forme tôt. Aucun problème particulier d'inondation n'est noté dans ce secteur, soit en absence ou en présence des glaces.

Le tronçon de Duberger

Le tronçon de Duberger s'étend du pont Marie de l'Incarnation jusqu'au confluent de la rivière Lorette. Favorisé par un écoulement lent en hiver (0,4 m/s) sur pratiquement toute sa longueur, les glaces se

LES DIFFÉRENTS TRONÇONS DE LA RIVIÈRE SAINT-CHARLES



forment rapidement sur ce tronçon. Ce couvert est stable et ne se brise que lors d'augmentation importante du débit. Généralement, ce couvert de glace fond sur place. Un affluent de la rivière Saint-Charles, soit la rivière du Berger, se déverse dans ce tronçon. Deux sites d'embâcle sont localisés sur cette rivière et le ministère de l'Environnement, a procédé à la construction d'un seuil et d'une estacade fixe permettant de résoudre ce problème. Un embâcle s'est aussi déjà produit au pont Marie de l'Incarnation, soit à la limite supérieure de la retenue. Un seul événement a été noté historiquement à cet endroit et aucune mesure corrective spéciale n'a été envisagée à cet endroit pour le moment.

Le tronçon Neufchatel - Les Saules

Ce tronçon s'étend de la rivière Lorette à la municipalité de Loretteville et peut être qualifié de tronçon à forte pente (10 m/km); l'écoulement y est torrentiel à maints endroits. Les glaces se forment difficilement sur ce tronçon et ne recouvrent pas toujours l'entière surface de la rivière, laissant à découvert de petits secteurs à écoulement rapide. Le couvert se forme à partir des rives et des obstacles présents en rivière. C'est un tronçon où une augmentation du débit provoque le départ des glaces en maints endroits. Il s'agit du tronçon où les inondations en présence des glaces sont les plus spectaculaires et dévastatrices. L'ouvrage décrit dans ce rapport a pour but de solutionner ce problème d'inondation.

Le tronçon de Loretteville

Le tronçon de Loretteville, qui s'étend de Loretteville au barrage de Château d'Eau, est caractérisé par sa forte pente. Le courant très rapide dévale chutes et cascades et plusieurs secteurs demeurent libres de glaces tout l'hiver. C'est une zone où la génération de frasil est importante, celui-ci s'accroche aux moindres singularités du lit. Le couvert de glace y est très instable et très sensible aux variations de débits.

La nature rapide de l'écoulement élimine le problème d'inondation à l'eau libre. Toutefois, la quantité importante de frasil généré en période de grand froid a causé, à maintes reprises dans le passé, l'obstruction quasi-totale de la rivière un peu à l'amont des chutes de Loretteville. A cet endroit, un lit de rivière embarrassé de gros cailloux, d'îlots de roche et même d'un pilier de pont servait d'amorce à la formation d'embâcle de frasil. Depuis le nettoyage du lit de la rivière et l'enlèvement du pilier du pont, il y a quelques années, le problème semble résolu.

Le tronçon de Château d'Eau

Ce tronçon, qui s'étend du barrage Château d'Eau au lac Saint-Charles, présente une pente très faible. Lors de la formation du couvert, les vitesses sont faibles, si bien qu'un couvert de glace solide se forme

au début de l'hiver et y demeure stable toute la saison. Au printemps, ce couvert retient toutes les glaces en amont de Château d'Eau, lesquelles fondent à toutes fins pratiques sur place. Le comportement des glaces en aval du barrage de Château d'Eau n'est donc pas influencé par les glaces d'amont . Dans ce tronçon, il se produit des débordements importants en condition d'eau libre et en présence des glaces. Il ne s'y produit pas d'accumulation de glace en raison des faibles vitesses d'écoulement, mais les superficies affectées sont grandes en raison de la topographie plane du secteur. Heureusement, ce secteur est très peu urbanisé et seulement quelques résidences sont affectées.

Synthèse des travaux de protection

On constate donc que beaucoup de travail a été réalisé afin de diminuer les problèmes d'inondation en présence des glaces sur la rivière Saint-Charles. Un report des principaux sites historiques d'embâcles en zones urbaines sur une figure (figure 3) permet de constater que le travail de protection est à peu près complété, si ce n'est le site de la rue Marie de l'Incarnation. Il faut cependant noter que l'ensemble des interventions en amont devrait normalement atténuer de façon marquée ce problème. La suite du présent rapport se limite donc à l'étude du tronçon où l'ouvrage de rétention des glaces est implanté.

ANALYSE HYDRAULIQUE

Le régime hydrologique

Présentement, le ministère de l'Environnement du Québec opère une station de mesure des débits sur la rivière Saint-Charles. Cette station est située à 0,80 km en amont de l'embouchure de la rivière Lorette et porte le numéro 050904; elle est en opération depuis le début de 1969. Pour la période de 1964 à 1969, les données étaient recueillies à une station située à Loretteville. Ces données sont cependant de courte durée pour réaliser une étude statistique, si bien que l'évaluation des débits journaliers de crues pour de grandes récurrences est faite à l'aide d'une méthode régionale (Hoang, 1977). Le tableau 1 présente les débits de crue ainsi obtenus dans le secteur où l'ouvrage est construit.

Les débits ainsi estimés par la méthode régionale sont conservateurs puisque la présence du lac Saint-Charles permet un faible laminage du débit de pointe.

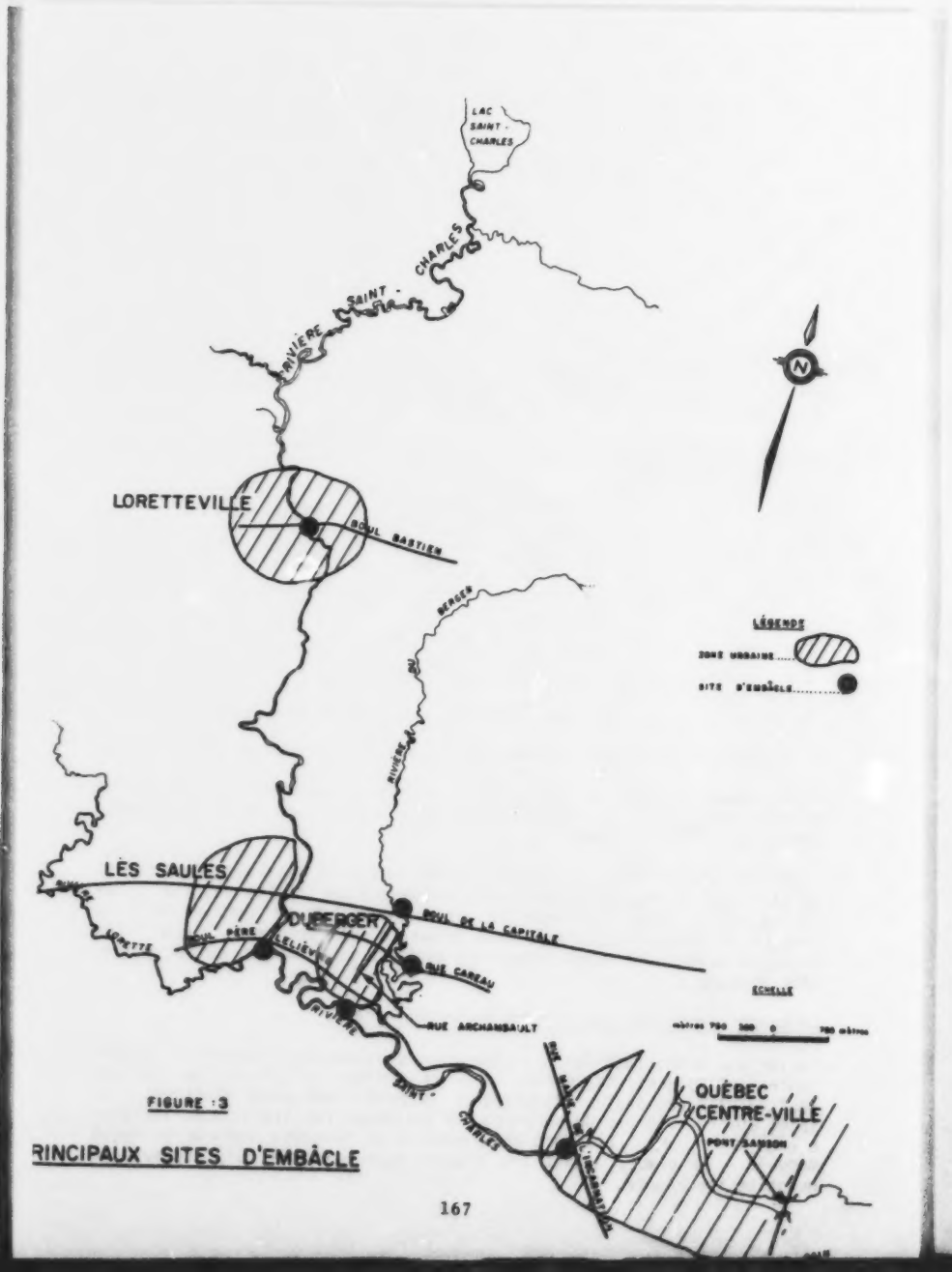


FIGURE : 3

RINCIPAUX SITES D'EMBÂCLE

Tableau 1: Estimation des débits journaliers de crue - rivière Saint-Charles (bassin versant, 357 km^2)

Période de retour (ans)	Débit (m^3/s)
2	95
5	127
10	148
20	170
50	197
100	220

Pour les besoins de notre étude, le tableau 2 fournit les débits maximaux journaliers pour chaque mois d'hiver, au site de la station 050904 (bassin versant de 357 km^2), ainsi que les débits moyens pendant ces mêmes mois.

Tableau 2: Débits maximaux journaliers et débit moyen pour les mois d'hiver

MOIS	DECEMBRE	JANVIER	FEVRIER	MARS
RECURRENCE (ans)	DEBIT (m^3/s)	DEBIT (m^3/s)	DEBIT (m^3/s)	DEBIT (m^3/s)
2*	9	3	3	9
5*	21	8	9	20
10*	33	11	11	28
Débit moyen mensuel	4,7	2,1	1,9	2,8

* Ajustement d'une loi log Pearson III sur les données de 1964 à 1980.

L'ajustement statistique est utilisé uniquement pour estimer des débits de récurrence égaux ou inférieurs à 10 ans en raison de la durée restreinte de l'échantillon.

L'analyse statistique des débits d'hiver permet de constater que ceux-ci ne font pas partie de la même population que les débits de crue. Les débits maximaux de récurrence 10 ans des mois d'hiver sont en effet de l'ordre du tiers du débit de crue annuel d'une récurrence 2 ans. Les crues se produisent généralement en avril et mai, moment où les glaces sont disparues.

Le comportement des glaces dans le secteur inondé

Le secteur présentant les problèmes les plus aigüs d'inondation en présence des glaces se situe dans la partie inférieure du tronçon Neufchatel - Les Saules (figure 2). Le profil est assez explicite à ce sujet et ne cause pas de surprise aux personnes familières avec les problèmes de glace. Il s'agit de l'endroit où la pente diminue de façon marquée. La glace, autant les glaçons que le frasil, s'y accumule en quantité importante.

En février 1981, une cinquantaine de résidences et une petite entreprise furent inondées. Lors de cet événement qui s'est produit le 21 février 1981, la hauteur de submersion atteignait 2,5 m à certains endroits. Suite à un hiver relativement froid, en l'espace de quelques jours, les températures se sont élevées à près de 10°C et des précipitations de l'ordre de 30 mm se sont abattues sur le bassin versant et sont venues gonfler les eaux de la rivière Saint-Charles. Le débit maximum a été estimé à 119 m^3/s. Il s'agit d'une estimation puisqu'il y avait un fort effet de glace au site de la station hydrométrique. L'accumulation de glace était appuyée sur le couvert solide recouvrant le tronçon à faible pente situé en aval de la rivière Lorette et l'embâcle d'une longueur de près de 1 kilomètre s'accompagnait d'une forte dénivellée.

Les dommages dus aux inondations en présence des glaces

Lors de l'événement de février 1981, les dommages aux résidences de ce quartier ont atteint 500 000$ et une petite entreprise a dû subir des pertes de 20 000$. Les dommages aux équipements collectifs et les mesures d'urgence ont été estimées à 80 000$ pour ce secteur.

Donc, la crue historique de 1981 a causé des dommages pour 600 000$. Toutefois, l'étude de rentabilité d'un projet nécessite une estimation sur une base annuelle des avantages découlant de son implantation. Pour ce faire, pour chaque secteur, une courbe de dommages d'inondation (D) est établie en fonction de sa probabilité de dépassement (P) qui est l'inverse de la période de récurrence. Un minimum de trois points est requis, soit l'évaluation de la récurrence du dommage zéro, de la récurrence du dommage associé à des crues historiques bien identifiées. Lorsque la topographie du site le permet, il est souhaitable de déterminer ce qui pourrait être une cote d'inondation absolue.

En appliquant le modèle développé à l'Ecole polytechnique (1981), soit un ajustement exponentiel de la forme suivante:

$P = Ae^{bd}$

P: Probabilité de dépassement
A et b: coefficients permettant l'ajustement de la courbe
$A>0$ et $b<0$
d: dommage associé à un événement

Le dommage annuel moyen (DAM) est représenté par la superficie sous la courbe dommage d'inondation en fonction de sa probabilité de dépassement et se calcule de la façon suivante:

$$DAM = \int_0^{\infty} Ae^{bd}dd$$

Lorsqu'il s'agit d'inondations par embâcles, le problème principal lorsque l'on veut représenter le dommage en fonction de la probabilité de dépassement réside dans le calcul de la récurrence à associer aux événements survenus ou potentiels.

Le niveau d'eau associé à un phénomène d'embâcle est fonction d'une multitude de facteurs très difficiles à isoler et surtout à quantifier. Parmi ceux-ci, les conditions météorologiques et hydrologiques au moment de la formation du couvert de glace, pendant l'hiver et au moment de la débâcle, la quantité de neige au sol, l'épaisseur du couvert de glace, sa cohésion, etc.

Er plus des difficultés énoncées précédemment, l'établissement de la fréquence dans ce cas est particulièrement difficile en raison du peu d'observations historiques. Le développement résidentiel d'une partie du secteur inondé est relativement récent (15 ans), si bien que la représentativité d'un tel échantillon laisse à désirer. On sait cependant qu'une inondation de moindre envergure s'est produite en 1974. Des modifications ont aussi été apportées à l'utilisation des sols du bassin. L'urbanisation se poursuit en amont du secteur inondé et une autoroute a été construite. Sans avoir des impacts majeurs sur le comportement de la rivière, ces ajouts peuvent modifier les récurrences. Pour l'application de cette méthodologie, le secteur inondé a été divisé en deux sous-secteurs, le dommage zéro correspond, selon les sous-secteurs, à des récurrences de 5 et 10 ans et l'événement de 1981 a été associé à une période de retour de 25 ans. Un dommage annuel moyen de 68 000$ a ainsi été trouvé pour ce secteur (Laganière, 1982).

LA SOLUTION

Trois types de solutions peuvent être envisagés pour régler un tel problème, soit les travaux en rivière, l'endiguement ou le rehaussement des secteurs inondés ou la rétention des glaces en amont.

Les travaux en rivière auraient exigé la création d'un bassin énorme afin de pouvoir accumuler les glaces sans créer de rehaussement de la ligne d'eau. La création d'un tel bassin ne pouvait s'intégrer au milieu et se serait avérée très dispendieuse.

L'endiguement ou le rehaussement des propriétés à protéger a aussi été rejeté. L'endiguement l'a été en raison des coûts, du peu d'espace disponible pour l'implantation de la digue, de la hauteur importante de cette dernière, ce qui la rend socialement difficilement acceptable. Le rehaussement des propriétés n'était pas réellement envisageable en raison des hauteurs de submersion.

La mise en place d'un ouvrage de rétention des glaces en amont du secteur à protéger a été choisie, d'autant plus qu'un site propice à l'implantation d'un tel ouvrage a été identifié à moins de 1,5 km en amont du secteur à protéger. Les berges de la rivière Saint-Charles sont à cet endroit escarpées de 10 mètres. L'axe de l'ouvrage est localisé à l'aval d'un court tronçon à pente plus faible. La rivière en amont de l'axe retenu présente aussi une largeur plus grande propice à la création d'une retenue de plus grand volume. Des observations

avaient aussi permis de constater que ce site constituait un champ naturel d'accumulation des glaces lors de la descente de ces dernières. Cependant, lors des fortes augmentations de débit (comme en février 1981), cette retenue se vidait de ses glaçons. Le but de l'ouvrage est donc d'assurer la rétention des glaces sur ce site. L'ouvrage est constitué d'un barrage rehaussant le plan d'eau et d'une estacade fixe. La hauteur du barrage et le dimensionnement de l'estacade ont été fixés principalement à partir d'expériences antérieures, soit celle du barrage Sartigan sur la rivière Chaudière, celle du barrage de Terrebonne sur la rivière des Mille Iles et celle de Saint-Raymond sur la rivière Sainte-Anne.

La théorie que nous connaissons concernant le comportement des couverts de glace ne s'applique pas à d'aussi petites rivières, aussi nous nous sommes limités à effectuer quelques calculs permettant d'estimer la stabilité du couvert à l'amont de l'ouvrage. Cet ouvrage crée un plan d'eau de 600 m de longueur et de 60 m de largeur.

La simulation du comportement du couvert de glace s'est effectuée en utilisant les débits normaux d'hiver pour la formation. Le couvert mince formé dans la retenue s'épaissit selon la rigueur de l'hiver et sert d'appui à l'embâcle. Des tests sont effectués pour différents débits. Il ne s'agit pas ici de simuler le comportement des glaces sur l'ensemble du tronçon, mais plutôt à l'aide d'hypothèses concernant le débit, l'épaisseur et la cohésion du couvert d'estimer la stabilité du couvert dans la retenue.

Dimensionnement de l'ouvrage

A partir de l'expérience acquise sur les autres rivières où des ouvrages du même type ont été construits et des calculs cités précédememnt, un ouvrage permettant de hausser le niveau d'eau de 2,5 m fut choisi (figure 4). Le plan d'eau ainsi créé ralentit suffisamment les vitesses de l'écoulement pour permettre la formation de couvert stable et résistant.

L'espacement entre les piliers servant d'estacade fixe a été plus influencé par les coûts de construction que par les besoins pour la rétention des glaces. Notre expérience avec les ouvrages cités précédemment (Sartigan, Terrebonne et Saint-Raymond), suggérait plutôt un espacement de 6 m entre les piliers; cette distance donne entièrement satisfaction pour la rétention d'un couvert de glace de dimensions semblables. Le barrage sur la rivière Saint-Charles devant être muni d'une passerelle et de vannes, il est apparu plus économique de diminuer l'espacement entre les piliers constituant l'estacade. Ainsi, les quatre piliers sont espacés de 3,5 m, ce qui fournit une crête déversante de longueur suffisante (17,5 m) pour l'évacuation d'une crue centennale avec un niveau d'eau amont acceptable.

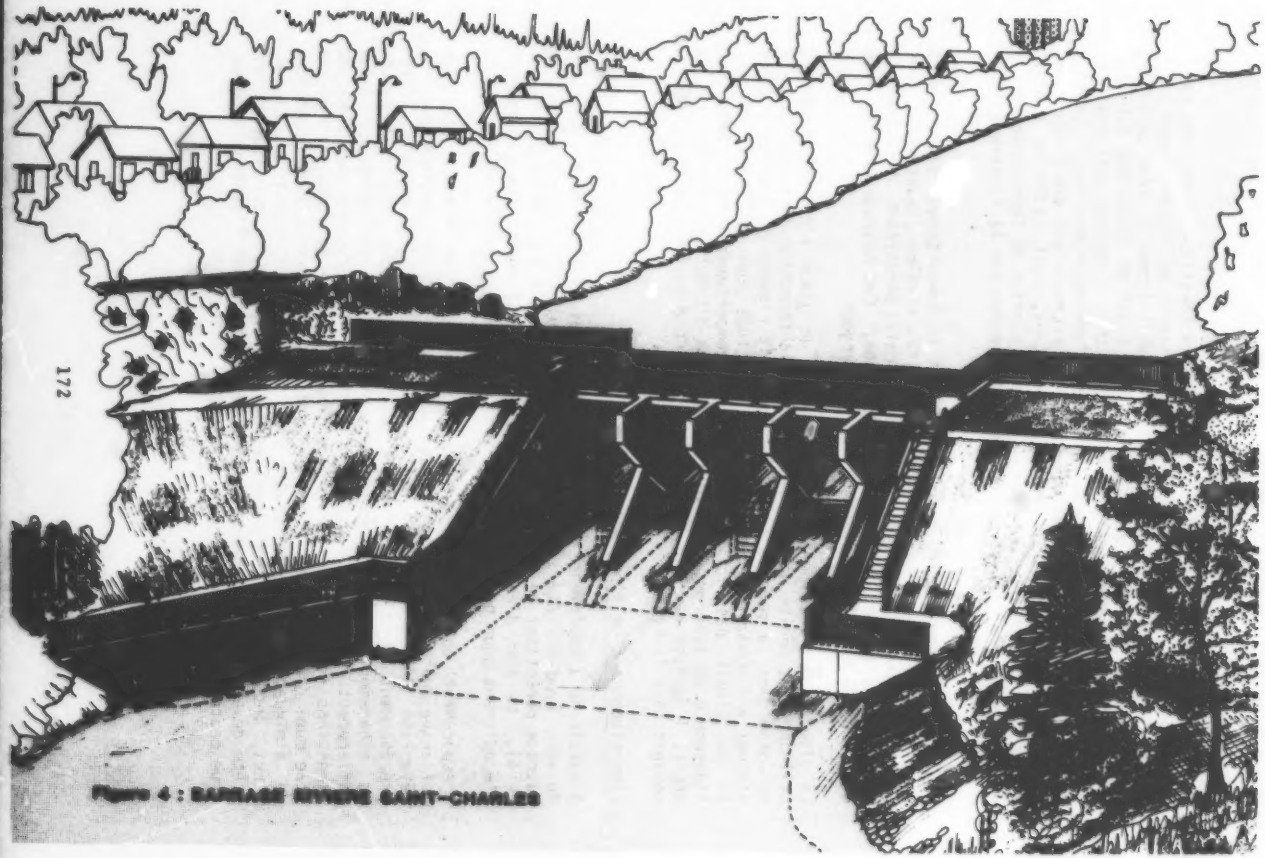


Figure 4 : BARRAGE RIVIERE SAINT-CHARLES

La présence des vannes et des poutrelles n'est pas non plus jutifiée pour la rétention des glaces. Leur mise en place au lieu d'un seuil fixe a été choisie de façon à permettre de vider la retenue en période estivale, ce qui entraînait moins de conséquences écologiques négatives. En période d'étiage estival, le réchauffement des eaux en plus d'affecter sa qualité aurait permis la prolifération des plantes aquatiques. Une sédimentation éventuelle risquait aussi de se produire si le plan d'eau était maintenu continuellement. L'ouverture des vannes permet aussi l'élimination de la passe migratoire, type d'ouvrage que les humains aiment bien, mais que les poissons n'apprécient pas toujours.

Le sol qui sert d'assises à l'ouvrage est constitué d'un schiste désagrégé offrant une bonne capacité portante. Un calcul de stabilité a aussi été exécuté afin de vérifier que l'ouvrage n'avait pas tendance à se renverser. De plus, pour assurer la sécurité contre le glissement, deux clefs ont été placées sous le radier de l'ouvrage.

La réalisation de l'ouvrage a coûté finalement un peu plus de 590 000$.

Analyse bénéfice-coût

Les bénéfices annuels associés à la protection contre les inondations ont été estimés antérieurement à 68 000$ et les coûts de construction se sont chiffrés à 590 000$. En supposant que les coûts d'exploitation et d'entretien sont de l'ordre de l'ordre de 2 pour cent du coût annuel d'immobilisation, que la vie économique du projet est de 50 ans et que les taux d'intérêt se situent à 10 pour cent, il est possible de connaître le coût annuel.

Coût annuel d'immobilisation	59 500$
Entretien et exploitation	1 200$
Coût total annuel	60 700$

La comparaison du bénéfice escompté, soit la disparition des dommages et du coût, permet d'établir le rapport bénéfice-coût; celui-ci équivaut à 1,12.

Le financement de ce projet a été réalisé grâce à la participation des trois paliers de gouvernement. Les gouvernements fédéral et provincial ont contribué pour chacun 45 pour cent du coût total, alors que la ville de Québec a fourni les derniers 10 pour cent. La construction a été réalisée à l'automne 1983 et l'ouvrage était opérationnel au cours de l'hiver 1983-84.

Comportement de l'ouvrage au cours du dernier hiver

Il est évidemment bien difficile d'évaluer le comportement d'un tel type d'ouvrage à partir des observations réalisées au cours d'un seul hiver. Les informations recueillies fournissent cependant une image intéressante.

Dans la partie horizontale de la retenue (600 m), le couvert de glace s'est formé très rapidement au début de décembre, puis a progressé vers l'amont sur environ 300 m supplémentaires, simultanément la retenue se remplissait de frasil et le niveau dans la partie amont de la retenue se rehaussait de plus de 1 m au cours du mois de décembre (figure 5). La glace se présentait en couche et dans la zone d'éclaircie amont de la glace de fond était visible. Le débit en décembre a varié de 2,5 à 20 m^3/s tout comme les conditions météorologiques qui furent très variables, neige, pluie verglaçante et températures oscillant entre +2 et -25°C.

Sur le tronçon amont, la surface à l'eau libre s'est rétrécie graduellement à partir de la glace de rive et de glace formée sur les obstacles en rivière, roches, îlots rocheux. La couverture n'était pas complète, mais les éclaircies restantes étaient très courtes (10 m) et étroites (1 à 2 m).

Les éclaircies avant leur rétrécissement ont cependant été suffisantes pour générer suffisamment de frasil pour remplir la retenue, un volume de 50 000 m^3 de frasil s'est effectivement accumulé dans la retenue.

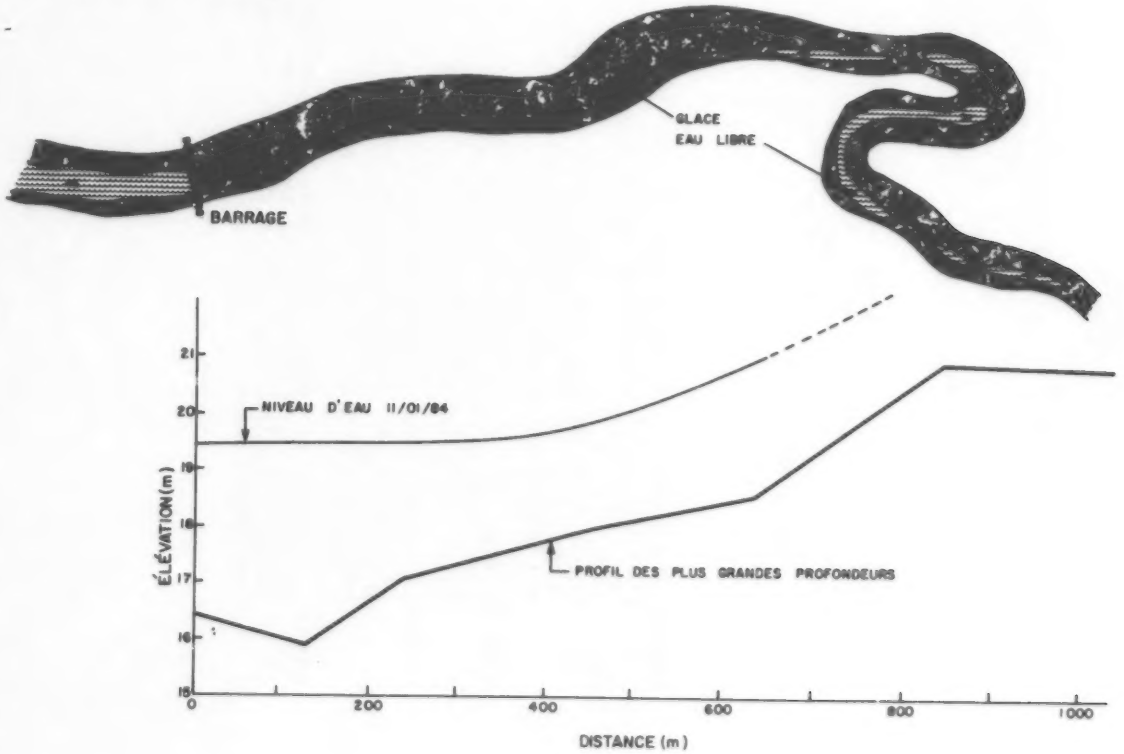
A l'aval immédiat de l'ouvrage, la zone d'éclaircie s'est graduellement recouverte de glace au cours du mois de janvier.

Des observations ont aussi été réalisées plus loin en aval de l'ouvrage, soit dans le secteur assujetti aux inondations. Au cours du dernier hiver, aucune accumulation de frasil n'a été notée contrairement à l'année précédente où une accumulation importante de frasil s'était produite. Celle-ci s'étendait sur plus de 6 kilomètres et présentait une épaisseur variable de quelques centimètres à 1,5 mètre pour un volume estimé de 110 000 m. La diminution de frasil générée ne peut être totalement attribuée à la présence de l'ouvrage. Les conditions climatiques et hydrologiques ont été bien différentes. Le débit moyen était en effet plus élevé en décembre 1982 (13 m^3/s) et les fluctuations de température très rapides et importantes. L'ouvrage a cependant le mérite d'avoir retenu 50 000 m^3 de frasil et d'avoir restreint la surface génératrice.

Les conditions printanières ont été peu révélatrices du comportement des glaces dans des conditions critiques. Le débit s'est élevé graduellement tout comme les températures, si bien qu'aucun mouvement de glace n'a été réellement noté. Les glaces ont littéralement fondu sur place, tant dans le secteur sujet aux inondations que dans le tronçon amont de l'ouvrage.

Figure 5 : RIVIERE ST-CHARLES

(tronçon en amont du barrage)



CONCLUSION

Le travail qui vous a été présenté décrit sommairement le cheminement d'un dossier de protection contre les inondations en présence des glaces. Le design de l'ouvrage a été conçu principalement à partir de l'expérience acquise sur d'autres ouvrages de rétention des glaces. Il s'agit cependant d'une première en ce sens que c'est la première fois qu'un tel ouvrage est mis en place par le ministère de l'Environnement sur une aussi petite rivière.

La campagne d'observation des glaces entreprise cet hiver sera poursuivie au cours des prochaines années et ce projet servira afin de parfaire notre expertise sur le comportement des glaces des petites rivières. On sait que le Québec est parsemé de rivières de ce genre et que les problèmes d'embâcle y sont relativement fréquents.

BIBLIOGRAPHIE

BOUILLON ANDRE, 1981. "Les inondations sur le territoire de la Communauté Urbaine de Québec". Service d'aménagement du territoire, Communauté Urbaine de Québec, Québec.

ECOLE POLYTECHNIQUE DE MONTREAL, 1981. "Estimation des dommages moyens causés par les inondations". Rapport préparé pour le ministère de l'Environnement du Québec, projet CDT, P. 264, Montréal.

HOANG Van Diem, 1977. "Estimation des débits journaliers de crue printanières des rivières du Québec méridional". Publication H.P.-40 du ministère des Richesses naturelles, Québec.

LAGANIERE, Marcel, 1982. "Etude des glaces sur les rivières Saint-Charles et du Berger - Analyse avantages-coûts". Direction des Ouvrages hydrauliques, Ministère de l'Environnement, Rapport interne 0509-5B, Québec.

TREMBLAY, Albert-Réal et TREMBLAY, Odina, 1978. "Etude des glaces sur les rivières Saint-Charles et du Berger". Ministère des Richesses naturelles, Rapport interne 0509-3A de la Direction de l'Aménagement, Québec.

DISCUSSION

Doug Hodgins

If possible, could you provide information on the cost of the control structure?

Reply

The total cost of the ice control structure is about $590,000 and may be divided in this way.

1.-	Land clearing	$ 5,645.00
2.-	Excavation	16,670.00
3.-	Cofferdams and dewatering	40,000.00
4.-	Forms	51,624.00
5.-	Reinforcement steel	35,985.00
6.-	Concrete	117,736.00
7.-	Steel parts imbedded in concrete	10,470.00
8.-	Embankment	127,130.00
9.-	Racings, rip-rap, stairs, culverts, etc.	36,556.00
10.-	Stop logs	5,040.00
11.-	Gates and hoists	39,840.00
12.-	Drainage and landscaping	26,500.00
13.-	Mobilization, demobilization and coordination	80,000.00

R. Beauchemin

Is there control capacity at the discharge of lac St. Charles? Very little control can be had from the dam at the outlet of lac St. Charles to help the ice control structure downstream near blvd de la Capitale, because the dam acts at that time of the year as a submerged weir.

Reply

The dam at the outlet of lac Saint-Charles cannot be used to reduce the high flow because the capacity of storage of the lac Saint-Charles is very small. At this time of year the dam is also generally submerged by the downstream high water level so that the operation of this dam is not really effective.

Marc Drouin

Auriez-vous regardé la possibilité de construire un étranglement de la rivière au site du barrage de façon à retenir les glaces?

Réponse

La possibilité de construire uniquement un ouvrage permettant de rétrécir la rivière afin de retenir les glaces n'a pas été longuement regardée at ce pour diverses raisons. La construction d'un tel étranglement aurait certes contribué à améliorer la rétention des glaces dans ce champs naturel d'accumulation des glaces mais n'offrait pas suffisament de sécurité; l'augmentation très grande des vitesses dans cet étranglement aurait pu contribuer à déstabiliser le couvert amont et notre expérience de comportement des glaces n'est pas suffisante pour proposer un tel type d'ouvrage. Le plan d'eau permanent créé par cet étranglement n'est pas non plus désiré si bien que l'ouvrage avec des vannes tel que construit répond mieux à l'objectif de ne pas modifier les conditions en l'absence des glaces.

Pierre-R. Tremblay

Quelle est l'expérience d'operation de ce type d'ouvrage ailleurs, à Terrebonne en particulier, est-it déjà arrivé que l'accumulation de frasil à l'amont de la retenue ait causé des inondations?

Réponse

L'expérience que nous avons avec nos structures de contrôle des glaces montre des accumulations importantes de glace à l'amont des retenues, généralement ce phénomène ne cause pas de problème. Dans le cas particulier de l'ouvrage de Terrebonne sur la rivière des Milles Iles, il s'est effectivement produit un problème d'inondation sur l'île Saint-Jean à l'amont de la retenue en février 1981. L'ouvrage dont il est ici question est similaire à celui implanté sur la rivière Saint-Charles, un seuil d'une hauteur de 2,5 m créant un plan d'eau horizontal d'environ 600 m de longeur. A cet endroit, la rivière des Milles Iles se divise en deux et l'ouvrage est situé sur un seul embranchement. En 1981, la retenue était remplie de frasil avant le début de la crue at avec l'augmentation du débit, les glaces du tronçon amont sont venues

s'écraser à l'amont du couvert stable de la retenue, causant un rehaussement du niveau d'eau, ce qui provoqua une inondation sur l'île Saint-Jean. A ce moment, le débit de ce côté de l'île Saint-Jean était de 330 m^3/s et le débit total de la rivière de 1060 m^3/s. On ne peut toutefois préciser lors de l'inondation la part de responsibilité que peut avoir eu le frasil puisqu'il était impossible de vérifier si le frasil est demeuré en place lors de la crue.

Afin de régler ce problème d'inondation, le ministère de l'Environnement a procédé à un rehaussement de la berge dans le secteur sensible aux hauts niveaux engendrés par la présence des glaces. Il est aussi intéressant d'établir un parallèle entre l'ouvrage de Terrebonne et celui de la rivière Saint-Charles puisque les ouvrages créent des plans d'eau comparables. Les conditions sont cependant plus sévères sur la rivière des Mille Iles à Terrebonne en raison des débits plus importants susceptibles de s'y produire (débit du 22 février 1981, 330 m^3/s, récurrence 10 ans) que sur la rivière Saint-Charles (220 m^3/s, récurrence 100 ans), ce qui tend à prouver la sécurité de l'ouvrage mis en place sur la rivière Saint-Charles.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

ICE JAM RESEARCH NEEDS

R. Gerard[1]

for

Working Group on River Ice Jams
NRCC Subcommittee on Hydraulics
of Ice-Covered Rivers

ABSTRACT

Suggestions developed by the NRCC Working Group on Ice Jams for high priority research needs for ice jams are given. The suggestions concern ice jam formation, development and failure at freeze-up and break-up. It is not a literature review. Related processes such as frazil formation, hanging dams and ice deterioration were excluded from consideration. It is concluded that, despite significant progress in the past two decades, the work of developing a real understanding of ice jam fundamentals has really only just begun.

[1] Department of Civil Engineering, University of Alberta, Edmonton, Alberta, T6G 2G7. On study leave at USACRREL.

SUJETS DE RECHERCHE PRIORITAIRES SUR LES EMBÂCLES

par R. Gerard[1]

pour le

Groupe de travail sur les embâcles
Sous-comité du CNRC chargé de l'hydraulique des cours d'eau couverts de glace

RÉSUMÉ

Le document fait état des sujets de recherche prioritaires sur les embâcles qu'a identifiés le Groupe de travail du CNRC sur les embâcles. Les sujets se rapportent à la formation, à l'accumulation et à la rupture de l'embâcle, au moment de la prise de la glace et de la débâcle. Il ne s'agit toutefois pas d'un tour d'horizon de ce qui a été publié sur le sujet. Les manifestations connexes comme la formation du frasil, les bouchons de frasil (barrages suspendus) et la désintégration de la glace ne font l'objet d'aucun traitement. On conclut en disant que l'élaboration d'une explication véritable des mécanismes fondamentaux des embâcles ne fait vraiment que commencer, bien que des progrès sensibles aient été accomplis dans le domaine depuis vingt ans.

[1] R. Gerard est au Département de génie civil, University of Alberta, Edmonton (Al T6G 2G7. En congé d'études au USACRREL.

INTRODUCTION

An ice jam has been defined by IAHR (1984a) as "a forced accumulation of ice fragments in a waterway". This definition has been used as a guide herein - in particular it excludes consideration of hanging dams and ice dams.

Ice jams are an integral part of both freeze-up and break-up on a river. They can also cause considerable difficulties for man's activities, such as sudden disastrous flooding. The more important impacts of ice jams are described and illustrated in IAHR (1984a). Suffice it to say here that in recent years there has developed an increased awareness of the significant role of ice jams in hydrotechnical engineering in cold regions, and of the need to understand and consider them.

The current understanding of the phenomenon is poor and there is an urgent need to improve it. Of all countries, outside the U.S.S.R., Canada has the largest vested interest in understanding ice jams and should obviously play a leading role in the required research. To provide some guidance in mounting this research, suggestions made by the NRCC Working Group on Ice Jams as to the most important aspects of ice jams, and the needed research, are presented in what follows. Because of space limitations and the terms of reference of the Working Group, the discussion is confined to ice jams themselves, and does not consider the plethora of research needs on other apsects of river ice regime. It is also not a review of the literature on the subject. Such reviews have been provided by IAHR (1984b), Davar (1977) and others. Nor is it intended to be exhaustive. Instead the aim has been to highlight what are perceived by the Working Group to be the outstanding problems and to point out that, although there is a common perception that a lot of ice jam research has been done and there is little more of practical value to be learned, this is far from the truth. In fact, the battle for a true, and practical, understanding of ice jams has hardly been started.

Ice jams occur both at freeze-up and break-up. While there are obvious similarities in the jams formed in the two circumstances, there are sufficient differences to make it worthwhile treating them separately. Those features they have in common that require research will be only mentioned once.

FREEZE-UP ICE JAMS

On most rivers the formation of an ice jam is a necessary precursor to the development of an ice cover. As cold weather continues the water cools to 0°C, frazil ice begins to form and accumulate on the surface, and shorefast ice grows out from shore. Eventually, at some point along the stream, the width between the edges of the shorefast ice is small enough, and the concentration of floating frazil ice is high enough, to cause the frazil pans or slush to lodge, or jam, between the shorefast ice. Thereafter frazil moving down the stream accumulates upstream of this lodgement site and an ice jam develops, forming the initial ice cover on the stream.

Lodgement

This is possibly the most important feature of freeze-up jams, both because it is responsible for initial ice cover formation and because it is likely very sensitive to a change in flow regime, such as that which

accompanies dam construction. Yet it is a feature about which little is known. Lodging of frazil between shorefast ice obviously depends on channel geometry, frazil concentration and form, open water width, flow and meteorological conditions. A typical situation just prior to lodgement is shown in Figure 1.

Figure 1. Looking downstream over frazil pans moving through surface contractions caused by shorefast ice.

The most pressing need is for systematic documentation of such field situations to define, or at least place bounds on, the conditions necessary for lodgement. Laboratory and analytical studies of the interaction between slush, pans and floes in such constrictions could provide useful supplementary information on the important processes involved. Most work to date has involved laboratory studies of the limitations to passage or entrainment of solid ice floes at contractions. Although the primary application of such studies is to lodgement at freeze-up, this application is limited because of the neglect of the frail nature of frazil slush and pans and their mechanical properties under various meteorological conditions.

Accumulation Progression

Following lodgement, the ice moving down from upstream accumulates behind the lodged ice if the velocity is not too high. The nature of this accumulation depends strongly on its bulk compressive strength, the effect of

the flow under the accumulation, and the velocity required for entrainment of the approaching frazil ice at the head of the accumulation and for its subsequent deposition.

Because they provide indirect indications of the above parameters, the need is for field measurements of the character of accumulations in such freeze-up jams, such as the accumulation thickness, porosity and roughness along the jam; the manner of development of the accumulation, such as the geometry of the head of the accumulation, and the packing and shoving of the ice in the accumulation; degree of refreezing and so on. Direct measurements of these features will obviously be easier in small streams but, because of the simpler analytical situation, measurements on large streams are also necessary. In addition to direct measurements, some of the above features can be deduced indirectly, at least approximately, from simple measurement and analysis of the water surface profile. Again, a valuable supplement to these field measurements would be laboratory studies of the fundamentals of the processes involved (as against physical models of given situations). Such investigations may or may not use ice. For example, much of the current understanding of ice jam processes came from early work on log jams using wood pieces in the laboratory. One very important feature of accumulation strength - the effect of a thin refrozen layer - could most usefully be studied in the laboratory. Other features that could be studied in the laboratory are the hydraulics of shallow flow beneath such rough accumulations, the erosion and deposition of frazil in its various forms moving under an accumulation, the possibility of thickening of the accumulation by shoving causing the accumulation to ground and, indeed, the effect of features of a non-prismatic channel on development of the accumulation in general. As usual, such field and laboratory studies will benefit from the development and application of related analytical models, either theoretical or empirical, to summarize the findings.

Hydraulics

Mention was made above of the need for study of the hydraulics of flow under ice jam accumulations. Progression of the accumulation causes the flow to be unsteady. There is likely to be a strong interaction between accumulation development, the water profile and the flow distribution along the accumulation. The current understanding is only of accumulation development under a steady approach flow. Very little is known, for example, of how a developing accumulation adjusts with an unsteady inflow, particularly if this approach flow is subject to sudden and significant changes such as might be expected downstream of a peaking hydroelectric plant.

BREAK-UP ICE JAMS

Ice jam initiation at break-up is usually quite different from that at freeze-up. It is felt break-up of an ice cover while it is still competent can only be accomplished by surges released by ice jam failure, and that most major break-up ice jams are simply the result of stalling of the break-up front at a resistant portion of ice or at a location, such as the head of a reservoir, that has a normal ice cover but has an expanding channel geometry that weakens the impetus of the surge. Hence, no 'lodgement' is generally involved for a break-up jam, the existing solid ice cover providing sufficient 'key' for the jam; the accumulation forms more rapidly and under much more

unsteady circumstances; and the accumulation consists of a cohesionless collection of sizeable strong chunks of ice, without the refreezing complication present at freeze-up.

Jam Initiation

As for freeze-up, the major, yet least understood, feature is jam initiation. As mentioned this usually involves the stall of a break-up front against a resistant solid ice cover rather than the lodgement mechanism characteristic of freeze-up jams. The initiation of a break-up jam therefore has more to do with the formation of such resistant portions of the ice cover, and the characteristics of the surge that is responsible for break-up, than with the passage of fragmented ice through various constrictions and floe entrainment that has been the focus of research on ice jam initiation in the past.

A major need is observation of break-up progression in the field over long reaches, the identification of the major ice jams, and the reasons for their formation. A related item is to determine how strong an ice sheet needs to be to stall break-up in various circumstances. This should be amenable to laboratory study, although it will require development of appropriate scaling laws and use of a model 'ice' that correctly scales the salient mechanical properties of the ice cover [see, for example, IAHR (1984b)].

Jam Development

There is evidence that break-up jams can develop in a variety of ways. The most elementary, but perhaps the rarest as far as severe jams are concerned, is the quasi-steady accumulation of single-layer floes moving downstream on a more or less steady flow, as usually occurs at freeze-up. The other, and probably more common, situation is the arrival of a "slug" of ice - more the cessation of a moving ice jam - in the presence of very unsteady flow. There has been no published investigation of whether jams formed under such circumstances would take up a geometry similar to that usually assumed. The formation of thicker, and sometimes grounded, accumulations would seem likely under such circumstances. Simple laboratory experiments using simulated ice fragments, and perhaps some theoretical analysis, should provide valuable information on this possibility. As they become available, field observations will better define the range of circumstances under which ice jams form and the appropriate initial conditions so that such laboratory and analytical studies can define the limits of the current quasi-steady formation assumption, and how best this theory might be modified to have wider application.

The distinguishing features of a break-up jam are illustrated in Figure 2. Evident is the downstream ice cover that resisted break-up, the fractured ice sheet that forms the transition between the solid-ice and the quite sudden change to the very fragmented ice that forms the body of the jam. An important question is what the ice cover transition from solid ice sheet to open water looks like in profile, as the maximum water level produced by the jam depends strongly on thisice cover profile. In the current absence of an abililty to measure accumulation thickness directly, field measurement and analysis of water levels through the jam, and physical model studies using correctly scaled model ice, could provide information on this.

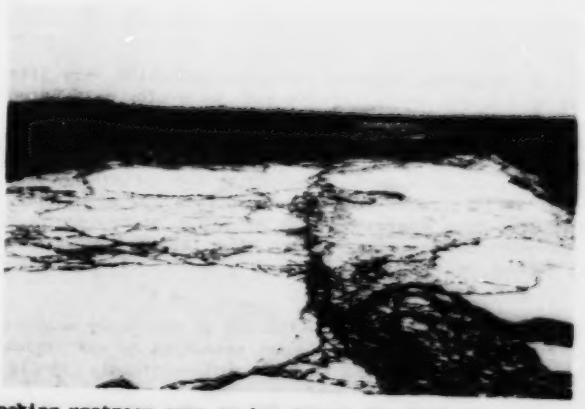


Figure 2: Looking upstream over an ice jam on the Mackenzie River near Port Norman, NWT

A related question is the ratio of the volume of ice in the jam to the original volume of ice in the contributing reach upstream of the jam. In some circumstances this ratio can provide an important constraint on the maximum size of an ice jam, and thereby on the maximum water level.

One important aspect of jam development that has had little investigation is the erosion and deposition of both ice and bed material under an ice jam. There is field evidence that bed scour under an ice jam can be considerable, and can have important implications with regard to pipeline and bridge pier design. In addition, erosion of ice fragments from beneath the jam as it thickens may be an important limiting process in jam development, for both freeze-up and break-up jams.

Another common limitation to jam development is flow diversion around the jam. After the water level reaches the floodplain, water (and perhaps ice) can move around the jam. This is a common occurrence in the field. Laboratory and analytical studies should be able to given some indication of how the ice jam responds once the accumulation begins to be lifted above the restraint of the banks and the proportion of the flow moving under the jam

decreases. Similar circumstances prevail in deltas when the backwater from a jam allows flow to move down secondary channels.

In the more settled regions of the country it is not uncommon for ice jam accumulations to become entangled with bridge superstructures. Although on large rivers this may be catastrophic for the structures, on small streams the structure will likely remain in place and contribute support to the ice jam. No procedure has been evaluated for determining the magnitude of this support. A related, and probably more common, question is the magnitude of the loads an ice jam can exert on bridge piers and superstructures. While these are likely not nearly as large as commonly imagined, some simple laboratory tests should help resolve the matter.

Jam Failure

Like ice jam formation, another feature about which very little is known is what causes an ice jam to fail. One mode is a slow melt brought about by warm weather and increasing temperature of the water moving down the river. However, what are far more perplexing and important are the conditions that will give rise to sudden failure of an ice jam. Useful contributions to understanding this problem could be provided by appropriate laboratory studies using model ice. A particular item of interest is how an ice jam responds to a surge moving down from upstream, such as would be caused by sudden heavy rain in the catchment, or sudden failure of a jam upstream. Presumably the response will depend in large measure on the nature of the "key" of the ice jam.

Sudden ice jam failure is usually the aim of emergency measures initiated after ice jam flooding begins. One common method is to use explosives to try to accomplish this but, as many case studies attest, it is not often successful. Another technique used to break up ice jams in navigation channels is "surging", using sudden releases from upstream storages. An important need is an investigation of such ice jam removal techniques with two aims: to determine the best approach in various circumstances and, perhaps more importantly, to better define the limits of what can be achieved and hence avoid the risk of life and property that often attends attempts at ice jam removal.

An understanding of ice jam failure is probably the most important and pressing problem with regard to ice jams. Almost no fundamental research has been done on the subject.

INSTRUMENTATION

Strong limitations on field research on ice jams are imposed by limited field techniques. Possibly the major problem is the current inability to directly measure ice accumulation thickness and strength. Although good indications of thickness can be obtained indirectly, such as from the thickness of shear walls left behind after ice jam failure, or from analytical simulation of measured water level profiles, direct measurement of accumulation thickness throughout an ice jam would remove many doubts and allow a major advance in the understanding of ice jams. It is commonly felt that, at least in large rivers, radar techniques may allow determination of jam thickness despite the necessary poor definition of the lower accumulation

boundary. Therefore, an important need is a thorough evaluation of the limitations of radar measurement of ice jam thickness.

Accumulation strength is another important ice jam parameter that to date has only been inferred. It would be most useful to measure this strength directly. One technique that has been suggested (D.D. Andres, Alberta Research Council, personal communication) is the use of pairs of piles or plates inserted into the accumulation from a bridge and then jacked apart. Obviously such an experiment would require substantial field support but it does seem feasible, particularly for freeze-up accumulations and measurements of the strengthening provided by the formation of a solid ice layer within the accumulation.

A fundamental need in any analysis of ice jam behaviour is knowledge of stage and discharge, and their variation in periods as short as an hour or so. The unreliability of standard discharge measurement techniques during freeze-up and break-up is legend. Even just reasonably reliable measurement of stage during such events seems to be beyond the capacity of the standard techniques used by the hydrometric measurement establishment. Because of this, available discharge estimates during break-up and freeze-up are based on extrapolation from other stations often far removed from the site of interest, so that even estimates of the mean daily discharge are frequently in error by a factor of 2 or more. The most fundamental need is therefore the development of a more reliable, but still relatively inexpensive, method of stage measurement, particulary for large rivers. A change in emphasis to measuring stage from above, rather than from below, may help in this. For example, remote recording of stage using time lapse (infrared?) photographs may be a reasonable alternative to the simple and effective, but manpower expensive, periodic surveillance by observers. Doubtless other techniques will be devised and applied once the importance of the problem, and the fact that the technique only has to be used for a limited period of time, is recognized.

Limitations imposed by the continually varying stage-discharge relationship during such events is generally much less than commonly perceived, and in no way negates the value of reliable stage measurement. Nevertheless, direct measurement of discharge during such events would be even more valuable. If a method of doing this could be developed, even if it is only applicable on small streams, it would be a very worthwhile contribution. However, the only relatively inexpensive techniques that come to mind and that may have some value are tracer methods. Other, much more expensive, possibilities might include ultrasonic or electromagnetic techniques, particularly for smaller streams.

MODELS

Physical Models

The use of scaled physical models has been suggested several times in the above. A review of scale-model relations and the state-of-the-art in model ice production suggests that such models are viable. Without doubt, because of the ease of handling it, the most convenient scale-model ice is ice itself, doped as necessary to get the needed reduction in strength, although its use does require a refrigerated laboratory. Considerable pioneering work on the use of such model ice has been carried out at the USACRREL laboratories (D.J.

Calkins, personal communication). However much remains to be learned, and the need for refrigerated facilities is a strong limitation. In lieu of this the possibility remains of developing satisfactory, non-proprietary, artificial model ice and investigating its use in model studies at room temperature.

Analytical Models

Availability of analytical solution techniques do not currently impose a limitation on the simulation of ice jam behaviour. Rather the limitation lies in the poor understanding of the ice jam processes, particularly ice jam formation and failure. Nevertheless computer simulation of ice jam formation and development, and the effect on discharge and stage, would help highlight the important weaknesses in the present understanding and, at least indirectly, by application and trial and error, assist in improving that understanding. Serious gaps will likely become apparent when trying to fit all the pieces together, but this in itself will make the exercise worthwhile. The fact that these gaps will, at present, have to be fitted with speculative, undocumented assumptions to complete the package will not be debilitating, as long as these, and the limitations they impose, are clearly identified. Such models would at least serve as a convenient summary and test of the state-of-the-art and, in this computer-oriented society, provide an effective means for technology transfer. Strangely, there is at present no non-proprietory computer model of ice jam formation and development, for either freeze-up or break-up. With regard to return for investment, the development, publication and servicing of such models would seem to be an important need in ice jam research. Every effort should be made to encourage an appropriate Government research agency to undertake such development, as only a Government agency can ensure the necessary non-proprietary status of the model and provide the required long-term servicing of it.

Another worthwhile research topic would be an empirical study of the hydrometeorologic conditions necessary for major ice jam formation. Such a study would be based on an analysis of historical data on ice jam formation at selected sites, and involve relating ice jam occurrence to pertinent meteorological data available for the same period. A few studies of this type have been published and have generally been successful in discerning at least approximate cause and effect relations. A thorough review of such studies would be a worthwhile study in itself.

SUMMARY AND CONCLUSIONS

Ice jams are a subject of increasing concern. Canada has a large vested interest in the subject and should obviously play a leading role in the necessary research. Despite the progress over the last two decades, development of a real understanding of ice jam fundamentals has only just begun. From the multitude of possible ice jam research topics, only some of which have been mentioned above, probably the most important, in terms of plausibility and return for investment, would be:

* Systematic field observation and measurement of ice jam behaviour. It is important that these be combined with observations of freeze-up or break-up, as appropraite, over a long reach of the river.

* Development of non-proprietary computer models of ice jam formation

and development at freeze-up and break-up, and their effect on discharge and stage, incorporating the present understanding of ice jam processes.

* Investigation, in the field, laboratory and office, of lodgement at freeze-up.

* Development of non-proprietory artificial model ice, and analytical and experimental assessment of scaling requirements for physical models of ice jams and break-up.

* Development and use of better methods of remote measurement of stage during freeze-up and break-up periods.

* Further laboratory and analytical investigations of the fundamentals of ice fragment and accumulation behaviour under steady and unsteady flow conditions.

* Laboratory investigations of the formation of ice jams under unsteady flow conditions. Modelling the mechanical properties of the solid ice cover will be an important component.

* Analytical and laboratory simulation of surge generation, propagation and decay in the presence of solid and fragmented (ice jams) ice covers.

ACKNOWLEDGEMENTS

This paper is presented as a synthesis of the various research needs suggested by the members of the Working Group on River Ice Jams of the Subcommittee on Hydraulics of Ice-Covered Rivers, Associate Committee on Hydrology, National Research Council of Canada. Members of the Working Group are D.D. Andres (Secretary), S. Beltaos (Chairman), B.C. Burrell, R. Halliday, F.E. Parkinson, S. Petryk, T.D. Prowse and the writer. The support provided by USACRREL in the preparation of this manuscript is gratefully acknowledged.

REFERENCES

Davar, Z.K. (1977) River Ice, Jamming, Flooding and Related Considerations - A Selective Bibliography. Report on Water Planning and Management Branch, Inland Waters Directorate, Atlantic Region, Dartmouth, N.S.

IAHR (1984a) River Ice Jam Primer. Report in preparation by Working Group on Ice Jams, International Association for Hydraulic Research, (scheduled for publication as part of the Proceedings of the 1984 IAHR Ice Symposium, Hamburg, Germany).

IAHR (1984b) River and Lake Ice Engineering Manual, G.D. Ashton (ed.), International Association for Hydraulic Research (scheduled for publication by Water Resources Press, Fort Collins in 1984).

DISCUSSION

J. Ford, Project Engineer, National Water Research Institute.

Is there enough statistical information given with research results, such as correlation coefficients and confidence limits? This is important, even though it may seem futile at times.

Response: In general, no. In our quest to perceive and define relations between variables, little attention has been paid to defining the possible range of the empirical parameters usually needed in these relations. In the past the scarcity of field data on ice jams has precluded this. However, enough data now seems to exist to warrant serious consideration of this very practical requirement. In a design situation it would seem vital to have some information on how much variation from the mean trend can be expected. The discusser may be interested to know that the writer made a similar point in a recent paper on streambed scour ("Field observations of scour on a small sand bed river", Proceedings of the Workshop on Bridge Hydraulics, R.A. Harrington and R. Gerard (eds.) Banff, Alberta, 1983, p.86).

D. Poulds, Consultant

How do water levels caused by ice jams compare to the 1:100 year flood levels currently used for flood plain mapping? Will the Working Group draw attention to, and make proposals for, the analysis of flood levels caused by ice jams?

Response: As Mr. Poulds implies by his question, there are very many sites in Canada that are prone to ice jam formation. At these sites it is not uncommon for the 1:100 year open water flood levels, that are currently the major concern in flood plain mapping across the country, to be almost inconsequential compared to more frequent flood stages caused by ice jams that can far exceed the open water levels. Yet generally little attention is paid to the ice jam stages. However, as methods of allowing for ice jams in flood frequency studies are developed and the word is passed around at workshops like this, the situation is changing. For example, the forthcoming 'Design Flood Guide for Canada' being assembled by the NRCC Associate Committee on Hydrology will have a section outlining how ice-related flood stages might be worked into routine flood frequency studies. On the other side of the border USACRREL are also putting together a report on 'Flood Frequency Analysis in Cold Regions'. In view of these efforts it is felt the Working Group will likely confine its attention to the physics and/or engineering of ice jams to provide the deterministic background for the above statistical studies, and the techniques for mitigation of the ice jam flood problem.

F. Parkinson, La Salle Hydraulics Laboratory (LHL)

Except for the lodgement mechanism, physical modelling of break-up jams is quite well understood and, indeed, has been carried out for many years. For example, LHL carried out a complete simulation of staging, accumulation and release of the ice at the Limestone cofferdam site on the Nelson River in Manitoba. The details of this study and its success were presented by Rick Carson at the 1982 Edmonton Workshop on the Hydraulics of Ice-Covered Rivers.

With regard to mathematical models, La Salle has also been modelling break-up jams mathematically for some time and we feel we have achieved good simulations of field conditions. Again, though, it is difficult to predict lodgement and this must be imposed on the model. However, I think it is very difficult to transfer math models from one operator to another.

B. Michel, Universite Laval

The author has given a very important paper on research needs in the study of ice jams. He has covered all aspects well and gives a good indication of what needs to be done in the future.

However we would take exception to what has been said in the paper about model studies. We do not believe that computer models should be given a priority as such in this field. What is really needed is a very good understanding of all basic physical processes and how they interact with each other. Given this, flowcharts can be prepared that would allow a computer programmer to easily prepare the necessary computer software, or model, that would be compatible with the hardware that is available.

The understanding of the physical processes cannot be obtained without field observations and measurements. We therefore believe that the first priority in ice jam research is more field data obtained in a systematic manner. Such data is badly needed.

In general freeze-up processes can generally be 'averaged' over the length of a river, and a simplified definition of the river reach geometry is usually sufficient to allow adequate definition of water surface profiles along the reach. This is why numerical analysis would normally be the best approach to solve problems related to this condition.

However the approach should be somewhat different for break-up jams. In such cases we think physical modelling is more appropriate. The ice floes have simple geometric characteristics that are easily simulated and the solid cover downstream of the jam can also be simulated, including its mechanical properties. Much information could be obtained in laboratory flumes and from models of specific sites. Such studies would significantly increase our fundamental understanding of break-up ice jams.

SESSION D

FLOW CONVEYANCE AND RESISTANCE

Chairman: Dr. B. Pratte; National Research Council of Canada

SESSION D

TRANSFERT ET RESISTANCE DES ECOULEMENTS

Président: Dr. B. Pratte; Conseil national de recherches Canada

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21,1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU-BRUNSWICK

ICEFORMS OF FLOATING LOOSE COVERS

S. P. Chee[1] and M. R. Haggag[2]

ABSTRACT

Just as in the case of bedforms in sediment transportation, floating loose covers have wavy underside configuration. This phenomenon has been observed in ice covers in nature. It has been argued that the formation of undercover iceforms is caused both by the flow and bed boundary conditions as well as the thermal effect. This paper explores the evolution of the underside shapes of buoyant loose ice covers from the point of the hydraulic effects. Differential equations governing the flow were derived which take into account the effect of sediment bedforms in influencing the shape of the undercover configuration. Verification of the theory was undertaken using laboratory flumes with polyethylene plastic pellets which have specific gravities close to natural ice to simulate the floating loose cover.

Juste comme les lits lors du transport des sédiments, les couvertures flottantes ont une forme ondulatoire dans leur partie inférieure. Ce phénomène a été observée pour des couvertures de glace dans la nature. Il a été argumenté que la formation de cette configuration intérieure de la glace est due à la fois à l'écoulement et aux conditions de frontière aussi bien aux effets thermiques. Cet article explore l'évolution des formes de la partie inférieure des couvertures de glace du point de vue des effets hydrauliques. Les équations differentielles gouvernant l'écoulement étaient dérivées et tiennent en compte l'effet des sédiments qui composent le lit ainsi que leur influence sur la forme des couverture de glace. La vérification de la théorie a été faite en utilisant des canaux de laboratoire. Des boulettes de polyethylène dont le poids spécifique est semblable à celui de la glace naturelle ont été utilisées afin de simuler le flottement des couvertures de glace.

[1]Department of Civil Engineering, University of Windsor, Windsor, Ontario.

[2]Formerly, Research Assistant, Department of Civil Engineering, University of Windsor, Windsor, Ontario.

INTRODUCTION

It is well known that the underside configuration of ice covers in rivers have wavy forms (Carey 1966, 1967, Larsen 1966, 1973, Tesaker 1970 and Ashton and Kennedy 1972). Researchers have attributed these iceforms on the bottom of ice covers to be caused both by hydraulic as well as by thermal effects. This paper deals with the theoretical derivation of the differential equations for the formation of these undulations in a loose buoyant cover based on a hydraulic analysis.

THEORETICAL ANALYSIS

The theory is developed based on a loose floating cover which can self-adjust to reflect the shape of the bedform and flow characteristics of the stream. The underside configuration of the buoyant cover is thus controlled by the bed boundary shape as well as the hydraulics of the flow.

General Equations

The symbols used in the derivation are shown in Fig. 1. At any section along the channel, the flow rate per unit channel width, q, is defined by the continuity equation

$$q = VY \tag{1}$$

where V and Y are the flow velocity and depth, respectively. Differentiating Eq. 1 with respect to x yields

$$dq/dx = V\, dY/dx + Y\, dV/dx \tag{2}$$

The conservation of energy principle can also be written for the same section, as

$$E = \eta_b + V^2/2g + Y + S_g \eta_s \tag{3}$$

in which E = total energy at this section; η_b = height of the bottom surface from the horizontal datum; η_s = cover thickness; V = mean velocity of the cross-section. Differentiating with respect to x the slope of the total energy line, S_f can be expressed as

$$S_f = -\frac{dE}{dx} = -\left(\frac{d\eta_b}{dx} + \frac{d}{dx}\,\frac{V^2}{2g} + \frac{dY}{dx} + S_g \frac{d\eta_s}{dx} \right) \tag{4}$$

in which the rate of change of the velocity head equals

$$\frac{d}{dx}\,\frac{V^2}{2g} = \frac{V}{gY}\,\frac{dq}{dx} - F_n^2 \frac{dY}{dx} \tag{5}$$

and $F_n = V/\sqrt{gY}$ is the local Froude number.

Both field and experimental observations suggested the equality of the cover top surface slope and the general average slope of the channel bed. The height of the cover top surface above the datum, H, can be written as

$$H = \eta_b + Y + \eta_s \tag{6}$$

Differentiating with respect to x the slope of the cover top surface S_o will be

$$S_o = -\frac{dH}{dx} = -\left(\frac{d\eta_b}{dx} + \frac{dY}{dx} + \frac{d\eta_s}{dx}\right) \tag{7}$$

Equations (1) through (7) can be combined to obtain the general underside configuration relation in the form

$$\frac{d\eta_s}{dx} = \left(-S_f + (1-F_n^2)S_o - \frac{V}{gY}\frac{dq}{dx} - F_n^2\frac{d\eta_b}{dx}\right)/(F_n^2 - S_{g1}) \tag{8}$$

in which $S_{g1} = (1-S_g)$ and S_g is the specific gravity of the cover material. The flow depth can also be expressed in the form

$$\frac{dY}{dx} = \left(-S_f + S_g S_o - S_{g1}\frac{d\eta_b}{dx} - \frac{V}{gY}\frac{dq}{dx}\right)/(S_{g1} - F_n^2) \tag{9}$$

Equations (8) and (9) are the general equations that relate the underside configuration to the bedform and the different flow conditions. The solution of these equations requires the determination of the local friction slope S_f.

Energy Slope

The total energy loss is represented by the sum of the individual losses of each subsection. Hence,

$$S_f = S_{f1} + S_{f2} \tag{10}$$

in which S_f = total friction slope at any section; S_{f1} = friction slope in subsection (1) and S_{f2} = friction slope in subsection (2).

The friction slope can be expressed in terms of the boundary shear by the relationship

$$S_{fi} = \tau_i/\gamma Y_i \,, \quad i = 1,2 \tag{11}$$

in which τ_i = boundary shear; γ = unit weight of water; Y_i = flow depth for the respective subsection; i = 1,2 refer to the subsections. In order to compute the friction slope, an expression for the boundary shear is next developed.

Boundary Shear and Friction Slope

The Cover Subsection

Figure 2 defines the forces acting on a sectional element of the cover subsection. Applying the momentum equation to this element in the horizontal direction gives

$$P_{2x} - P_{2(x+dx)} - \tau_{2x} dx = \rho q_2 (V_{2(x+dx)} - V_{2x}) \tag{12}$$

in which P_{2x}, $P_{2(x+dx)}$ = total pressure per unit width at the upstream and downstream sections of the element, respectively; τ_{2x} = boundary cover shear; dx = length of the element; ρ = density of water; q_2 = unit discharge at subsection (2); $V_{2(x+dx)}$ and V_{2x} = mean velocities at the downstream and upstream sections of the element, respectively.

The total pressure per unit width, which is taken equal to the static pressure, is

$$P_{2x} = C_p \gamma Y_2 \left(S_g \eta_s + \frac{1}{2} Y_2\right) \tag{13}$$

in which C_p = pressure coefficient.

The continuity equation is

$$q_2 = Y_2 V_2 \tag{14}$$

Hence, the horizontal shear force acting on the cover can be expressed as

$$\tau_{2x} = -\left(\gamma C_p S_g Y_2 \frac{d\eta_s}{dx} + \frac{\gamma}{g} V_2 \frac{dq_2}{dx} + \gamma \frac{dY_2}{dx} \left(C_p S_g \eta_s + C_p Y_2 - \frac{V_2^2}{g}\right)\right) \tag{15}$$

The application of the momentum principle in the vertical direction will give

$$\tau_{2y} = \gamma S_g \eta_s \tag{16}$$

in which τ_2 is the vertical component of the cover force. The total cover shear is given by

$$\tau_2 = \tau_{2x} \cos \theta_s - \tau_{2y} \sin \theta_s \tag{17}$$

where θ_s is the slope cover underside. The following assumptions can be made for practical purposes

$$\cos \theta_s = 1 \quad \text{and} \quad \sin \theta_s = d\eta_s/dx \tag{18}$$

An expression for the subsection depth (Chee, Haggag and Wong 1980) is given by

$$Y_2 = Y\lambda/(1+\lambda) \tag{19}$$

in which Y_2 = cover subsection depth; Y = total flow depth; λ = hydraulic radii ratio = R_2/R_1.

The cover subsection friction slope is obtained by combining these relationships with Eq. (11) to give

$$-S_{f2} = S_g\left(C_p + \frac{\eta_s}{Y}\ \frac{1+\lambda}{\lambda}\right)\frac{d\eta_s}{dx} + \frac{V_2}{gY}\ \frac{1+\lambda}{\lambda}\ \frac{dq_2}{dx}$$

$$+ \frac{dY}{dx}\left(C_p S_g \frac{\eta_s}{Y} + C_p\lambda(1+\lambda) - \frac{V_2^2}{gY}\right) \tag{20}$$

The Bed Subsection

Following a similar derivation the friction slope for the bed subsection is given by

$$-S_{f1} = C_p S_g \frac{d\eta_s}{dx} + (1+\lambda)\left(1+S_g \frac{\eta_s}{Y}\right)\frac{d\eta_b}{dx} + (1+\lambda)\frac{V_1}{gY}\ \frac{dq_1}{dx}$$

$$+ \frac{dY}{dx}\left(C_p S_g \frac{\eta_s}{Y} + C_p + \frac{\lambda}{1+\lambda} C_p - \frac{V_1^2}{gY}\right) \tag{21}$$

The total friction slope as given by Eq. (10) is then the sum of Eq. (20) and Eq. (21).

EXPERIMENTAL INVESTIGATION

Polyethylene pellets with a specific gravity equal to 0.92, which is close to that of natural ice, were used as the loose cover. The pellets have a disc shape measuring 4 mm in diameter, 2 mm in core height, and 1 mm at the sides. The buoyant loose cover has the ability to self-adjust its configuration as dictated by the flow conditions and the bedform. A modified form of hook gauge was used to measure the thickness and wavy shape of the cover.

Various types of solid immobile bedforms fabricated from concrete and sheet metal on wood were employed.

The tests were carried out in 0.46 m and 0.15 m wide flumes. Suitable traps were installed in the flumes to prevent the plastic pellets from entering the pumping system.

DISCUSSION OF RESULTS

The presence of the bedform makes direct integration of Eq. (10) from the relationships given by Eqs. (20) and (21) difficult. Numerical integration, therefore, becomes necessary.

In the solution of the differential equation, the direct step method was adopted. In the application of this technique, the value of $d\eta_b/dx$ are assumed to be known along the channel. At the starting point (X_o), the flow depth (Y_o) is taken as known and the value of $(dY/dx)_o$ can be calculated and the expected Y at (X_o+dx) can be determined. Then, the equations developed earlier are used to predict the cover thickness. The solution progresses along the channel to obtain the underside configuration.

The results of the underside configuration of the cover with an immobile dune bedform is shown in Fig. 3. The measurements were made at 7.62 cm while the computations were made at 2.54 cm. The experimental and computed results were found to agree.

From the results it can be noticed that a uniform regular wave configuration existed. An increase in flowrate increases the cover thickness which can be attributed to stability requirements. The wave length also increased with the discharge.

CONCLUSIONS

Differential equations were developed to predict the configuration of floating loose covers in the presence of immobile bedforms. This hydraulic phenomenon goes partly towards explaining the presence of wavy forms in the underside of ice covers. In the case of ice covers, the underside iceforms are attributed both to hydraulic and thermal effects.

ACKNOWLEDGEMENT

The research grant provided by the Natural Sciences and Engineering Research Council Canada is gratefully acknowledged.

REFERENCES

ASHTON, G. and KENNEDY, J.F., (1972), Ripples on Underside of River Ice Covers. Journal of the Hydraulics Division, ASCE, HY9, pp. 1603-1624.

CAREY, K.L., (1966), Observed Configurations and Computed Roughness of the Underside of River Ice, St. Croix River, Wisconsin. U.S. Geological Survey Professional Paper 550B. Geological Research, pp. B197-B198.

CAREY, K.L., (1967), The Underside of River Ice, St. Croix River, Wisconsin. U.S. Geological Survey Professional Paper 575-c, Geological Survey Research, pp. C195-C199.

CHEE, S.P., HAGGAG, M.R. and WONG, Y.F., (1980), Channel Configuration and Streamflow. Proceedings, International Conference on Water Resources Development, Taiwan, pp. 651-660.

LARSEN, P.A., (1966), Head Losses Caused by an Ice Cover on Open Channels. Boston Society of Civil Engineers, pp. 45-67.

LARSEN, P.A., (1973), Hydraulic Roughness of Ice Covers. Journal of the Hydraulics Division, ASCE, HY1, pp. 111-119.

TESAKER, E., (1970), Measurements of Ice Cover Roughness and the Effect of Ice Cover on Water Levels in Three Norwegian Rivers. IAHR Symposium on Ice, Iceland.

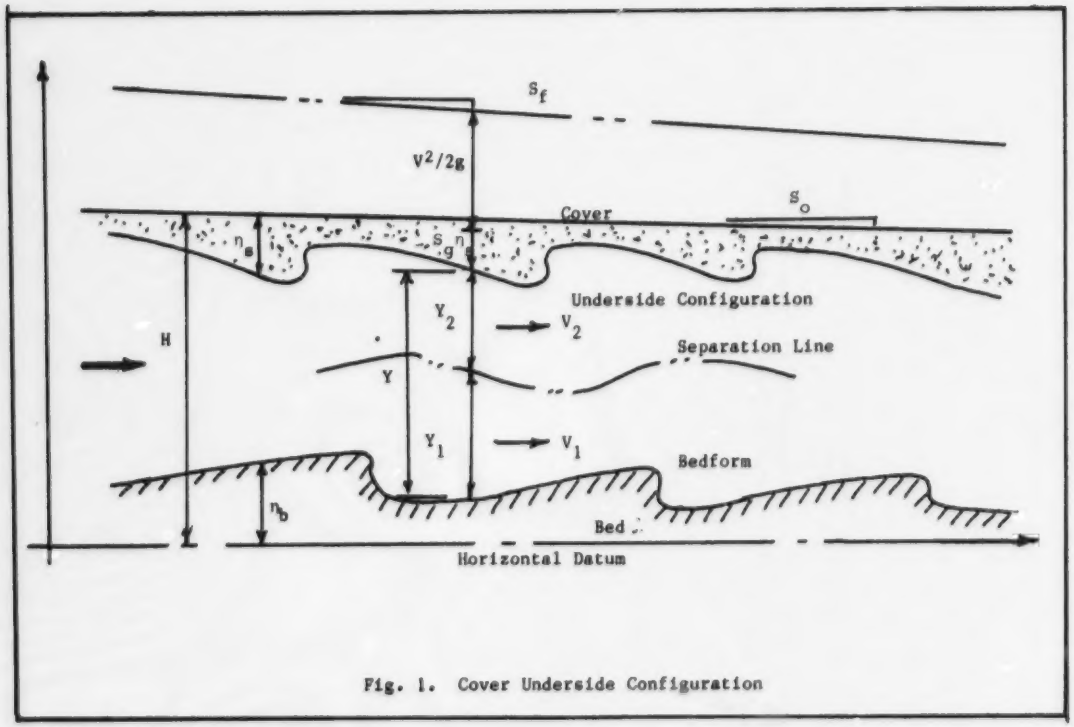


Fig. 1. Cover Underside Configuration

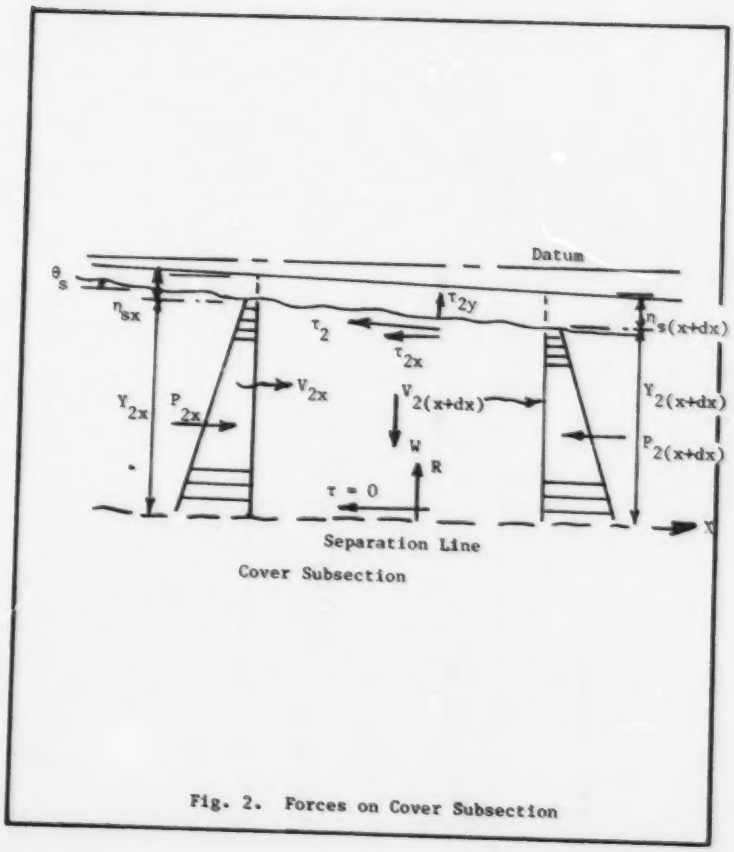


Fig. 2. Forces on Cover Subsection

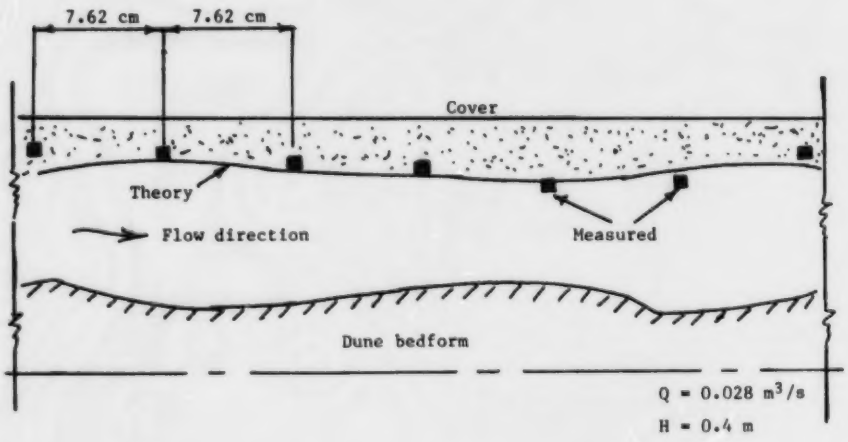


Fig. 3. Underside Configuration: Dune Bedform

DISCUSSION

James L. Wuebben

Did you examine the relationship between the phase of the ice form and bed form? That is, were they in phase, out of phase, or was there a phase lag?

If in fact you developed a wavy ice surface when the flume bed was plane, how could you determine if there was any direct relation between the two surfaces? Could not the wavy ice surface be simply due to its own instability and the general characteristics of the water flow? Since you had a well defined bed roughness it would have been interesting to have examined the inter-relationship between waveforms in more detail.

What was your range of flow velocities and how did you pick them? It would seem that your case where a wavy surface developed and no further pellet motion took place was rather a special case. Since your numerical model does not include ice pellet transport characteristics, that is fortunate, but in a real alluvial system you would have to account for continuing transport. Could you also comment on your range of energy slopes and roughness?

Reply

No a priori condition between the phases of the bedform and the underside cover configuration has been used in the development of the theoretical equations. Solution of the differential equations indicate a wavy underside cover for the cases of plane, dune and triangular bedforms. In the cases of the dune and triangular bedforms, examination of the plotted results indicate that the underside loose cover forms appear to be in phase with the bedforms.

Flow velocities range from 0.30 to 0.73 m/s. These velocities were not selected in advance. The polyethylene pellets were fed into the flow until a stable underside configuraion was obtained with sensibly no solid pellet transportation. The case of pellet (or the equivalent of ice in nature) and bottom bedload transportation has not been examined in this investigation. The range of energy slope varied from 0.005 to 0.035 and Manning's roughness ranged from 0.010 to 0.036 (bed) and 0.018 to 0.025 (cover underside).

D. I. Bray

Would you comment on the results of your experiments for a basic case of a plane bed rather than a duned rigid bed?

Reply

For the case of a plane bed, the equations can be integrated and give a wavy underside configuration. In the experiments with the plane bed, the plastic pellets assumed also an undulating underside shape that conforms sensibly to the analytical solution.

Pierre-R. Tremblay

What is the range of Manning n values for the bed (n_1) and for the ice cover (n_2) in your experiments? What are the thicknesses of the ice cover on the model and what would be the thickness on the prototype?

Reply

The range of Manning's n in the bed is 0.010 to 0.036 and for the cover underside it is from 0.018 to 0.025. The thickness of the model cover varied from about 5 cm to 12 cm. The study of the model scale relationship for extrapolating model cover results to the full-size has not been undertaken.

Nabil Elhadi

Generally, the dune-type configuration of the undersurface of an ice cover are observed at the early stages of ice formation and during the period before breakup while the ice undersurface has been observed to be smooth during the mid-winter period. Could you comment on this observation in view of the thermal as well as the hydraulic effects?

Reply

From our observations, since a smooth plane rigid bed could influence the formation of a wavy loose cover, it is, therefore, conceivable that the reverse could occur, that is, a rigid cover could cause the formation of dunes in a mobile bed. Our investigations were confined to

the hydraulic effects only and the thermodynamic influence has not been studied.

Sayed Ismail

Is the volume of the cover fixed? What is the effect of cover volume on the configuration of the cover?

Reply

The polyethylene plastic pellet cover which was used to simulate ice was loose and self adjusting. When the discharge increased, the thickness and hence the volume of the cover increased, but the configuration of the underside remained mainly unchanged.

Maurice Miron

In your experience, what is the maximum critical depth where the bottom dune has no effect on under floating form?

Reply

The largest flume which was used in the laboratory investigation has a height of 0.6 m. Up to this depth, the formation of an undulating underside profile still was possible, that is, the maximum critical depth was not yet attained. It would be most interesting to extend the experiments in subsequent investigations to determine the maximum critical depth for this case.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

COVERAGE COEFFICIENT FOR CALCULATING ICE VOLUME GENERATED

R. Hausser[4], J.P. Saucet[44] and F.E. Parkinson[444]

ABSTRACT

Calculation of the volume of ice formed in winter on an open water area on a river is done traditionally assuming that the whole open water area takes part in the thermal exchange. This approach overestimates the ice volumes since floating ice generated further upstream forms a partial protection of the open water surface, reducing the heat-loss.

An analytical derivation based on simple hypotheses is proposed which results in a "Coverage Coefficient" which takes this phenomenon into account. A few practical applications are used to show that the correction introduced to the traditional method can be large, reaching 40% in certain cases.

[4] *President*)
[44] *Senior Hydraulic Engineer*) *La Salle Hydraulic Laboratory, LaSalle, Quebec*
[444] *Vice President*)

RESUME

Le calcul du volume de glace formé en hiver sur une zone d'eau libre d'une rivière se fait traditionnellement en considérant que toute la surface libre participe aux échanges thermiques. Cette hypothèse conduit à surestimer les volumes de glace puisque la présence de glaces à la dérive, générées plus en amont, forme une protection partielle de la surface libre et modère la déperdition de chaleur.

Un calcul analytique basé sur des hypothèses simples est présenté qui permet d'introduire un "coefficient de couverture" rendant compte de ce phénomène. On montre par quelques exemples concrets que la correction apportée aux volumes calculés selon la méthode traditionnelle est souvent importante, puisqu'elle peut atteindre 40% dans certains cas.

INTRODUCTION

Calculation of the volume of ice generated on an open water river reach is traditionally done by applying a heat loss coefficient, Ka, between the water at 0°C and the colder air over the whole open area. The area participating in the heat exchange is generally corrected for the possible presence of shore ice, which can reduce significantly the volume of ice produced over a given period. We will show below how the presence of floating ice, formed further upstream, also reduces the active generating surface and ice production by a factor that can be significant in certain cases.

A simple analytical expression is proposed for a Coverage Coefficient which should be introduced into the calculations of river ice volumes generated.

REVIEW - CALCULATION OF ICE PRODUCTION WITHOUT TAKING INTO ACCOUNT THE ACTIVE SURFACE AREA REDUCTION COVERED BY FLOATING ICE

We will designate P(x) as the volume of ice produced on a river in a unit time between the 0°C isotherm and a cross-section located at distance x downstream. B is the mean width, and is assumed constant over the reach, and may be corrected for shore ice encroachment. Finally, Ta is taken as the absolute value of the air temperature, in °C.

The quantity of heat lost by the water to the air in a unit time is:

$$Q = Ka\ Ta\ B(x)$$

where Ka is the heat exchange coefficient between the water and the air in Kilocalories/m²/°C/day. Some values for this coefficient have been determined experimentally and are given in Table 1. A value of Ka = 460 Kcal/m²/°C/day has been found to be generally acceptable, considering light winds, since stronger wind increases the heat loss, and can significantly raise this coefficient.

The quantity of heat lost, Q, leads to the production of a volume, P, of ice:

$$Q = \rho\ L\ P\ (Kcal)$$

where ρ is the specific mass of ice 0.92 Kg/m³, and L is the latent heat of fusion, 80 Kcal/Kg.

We can find:

$$P = \frac{Ka\ Ta\ B\ x}{\rho\ L} \quad (m^3/day)$$

$$\text{or} \quad P = a\ x \qquad (1)$$

$$\text{with } a = \frac{B\ Ka\ Ta}{\rho\ L}$$

CALCULATION OF ICE PRODUCTION TAKING INTO ACCOUNT THE PRESENCE OF FLOATING ICE

Take C(x) as the proportion of the open water surface area covered by floating ice at section x.

Also, by definition the production of ice between sections x and x + dx during the time dt is:

$$\frac{dP}{dx}\ dx\ dt.$$

This volume is calculated the same way as in the preceding chapter, however taking into account the fact that the generating area is greatly reduced:

$$\frac{dP}{dx}\ dx\ dt = (1 - C)\ B\ dx\ \frac{Ka\ Ta}{\rho\ L}\ dt$$

$$\text{or} \quad \frac{dP}{dx} = (1 - C)\ B\ \frac{Ka\ Ta}{\rho\ L}\ . \qquad (2)$$

Now we can write that P(x) is the mass of ice crossing section x during a unit of time. A fixed observer at section x would see ice with mean thickness $\bar{h}$ passing at velocity V and covering a percentage C of the surface area:

$$P = B\ V\ C\ \bar{h} \qquad (3)$$

Calculation of the ice thickness is well known from earlier work, to which a thermal diffusion coefficient for ice (Kcal/m²/day/(°C/m) is introduced so the energy flux through a piece of ice of unit area and thickness h over a time dt is:

$$q = ki\ \frac{Ta}{h}\ dt$$

and this cooling causes a thickening dh:

$q = \rho L \, dh$

from which:

$$h \, dh = \frac{ki \, Ta}{\rho L} \, dt$$

and when integrated gives:

$$h = \sqrt{\frac{2 \, ki \, Ta}{\rho L} \, t}.$$

The ice pieces passing section x were formed somewhere between there and Section 0, and their thicknesses are between 0 and $\sqrt{\frac{2 \, ki \, Ta}{\rho L} \, \frac{x}{V}}$.

We will therefore take a mean thickness:

$$\bar{h} = \frac{1}{2} \sqrt{\frac{2 \, ki \, Ta}{\rho L} \, \frac{x}{V}}. \qquad (4)$$

Equations (2), (3) and (4) allow writing:

$$\frac{dP}{dx} + \frac{P}{\sqrt{x}} \, Ka \sqrt{\frac{2 \, Ta}{\rho L \, V \, ki}} = \frac{B \, Ka \, Ta}{\rho L}$$

$$\text{or: } \frac{dP}{dx} + b\frac{P}{\sqrt{x}} = a \qquad (5)$$

$$P(0) = 0$$

where: $a = \frac{B \, Ka \, Ta}{\rho L}$ (already defined after equation (1))

and $b = Ka \sqrt{\frac{2 \, Ta}{\rho L \, V \, ki}}$

Equations 5 have the following solution:

$$P(x) = ax \left(\frac{1}{b\sqrt{x}} - \frac{1}{2b^2 x} \left(1 - e^{-2b\sqrt{x}}\right) \right) \qquad (6)$$

Ice production calculated in the preceding section without taking into account the presence of floating ice can therefore be corrected by a coefficient:

$$M(x) = \frac{1}{b\sqrt{x}} - \frac{1}{2b^2x}\left(1 - e^{-2b\sqrt{x}}\right)$$

which can also be written:

$$M(y) = \frac{1}{y} - \frac{1 - e^{-2y}}{2y^2}$$

by considering the dimensionless coefficient:

$$y = Ka \sqrt{\frac{2}{\rho L ki}} \sqrt{\frac{Ta\ x}{V}}$$

or by substituting the values given earlier for ρ, L and Ka and taking ki as:

ki = 45.8 Kcal/m²/day/(°C/m) .... Michel (1978)

the coefficient becomes:

$$y = 1.2 \times 10^{-3} \sqrt{\frac{Ta\ x}{V}}$$

where: Ta is in °C
x is in metres
V is in metres/second.

We can also propose a simple expression for the concentration of ice pieces at a distance x from the 0°C isotherm. In fact, working with equations (3), (4) and (6), this becomes:

$$C(y) = 1 - \frac{1 - e^{-2y}}{2y} \qquad (7)$$

An indication of the variations of M and C as a function of y is given on Figure 1.

Values of y found in practical applications never exceed a few units as shown by the following hypothetical examples:

Case No 1: A short reach of a fast flowing river with moderate heat loss.

Ta = -5°C x = 100 m V = 5 m/s

From which we find: y = 0.012 and M = 0.99.

The presence of the floating ice only reduces the volume of ice generated by 1%.

Case No 2: A long reach of a slow flowing river with high heat loss:

Ta = -35°C x = 50 km V = 1 m/s

From which we find: y = 1.59, M = 0.44 and C = 0.70.

Therefore, 70% of the surface was covered with floating ice pieces 50 km from the 0°C isotherm, and the effective ice generation in the reach is less than half that which would have been found without taking into account the floating ice.

DEVELOPMENT AND APPLICATIONS

Like so many new developments in engineering, the need for the Coverage Coefficient became evident when calculations of a given known set of conditions did not give the same results as those measured in the field. In this case, the data was available for the Peace River downstream of Bennett Dam.

B.C. Hydro had gathered a unique data base over several winters including air and water temperatures, ice generation, cover build-up and retreat and water levels. This information was used to verify an existing ice model, and well identified differences were found. Closer inspection of the data and procedures suggested that the latter should be modified, leading to the first steps in the development of the Coverage Coefficient as described above. At the same time as this derivation was being done, other studies we had going on for the St. Lawrence River came up with similar discrepancies.

It was the combined data of these sources that made it possible to define the final presentation given above. Since then, this calculation procedure has been used successfully by the Lasalle Hydraulic Laboratory (LHL) on several recent studies. Two examples among the larger studies are discussed below:

Archipelago Project, St. Lawrence River, Montreal

For the last two years, LHL has been operating a large physical scale model of the St. Lawrence River at Montreal for Hydro-Quebec. The object of the studies was to help in determining the feasibility of developing the hydroelectric potential of the Lachine Rapids. Testing on the model reproducing winter ice conditions has suggested that it would be feasible to consider winter operation of the power-house with an open water reach upstream 9,000 m long. Velocities would be about 1.25 m/s for which calculations with two temperatures give:

Ta = -10°C	y = 0.32	M = 0.81
Ta = -30°C	y = 0.56	M = 0.71

It was therefore necessary to reduce the amount of ice considered with respect to the volume calculated by the traditional method. On the model, 20% to 30% less ice was introduced at the upstream end in the form of polyethylene grains or powder which have the same specific gravity and certain mechanical properties as ice.

Liard-Mackenzie River System - British Columbia

LHL carried out a large scale study for B.C. Hydro and Power Authority on the winter regime of these two rivers. The study included field observations, then development and operation of a mathematical model describing the freeze-up, ice cover evolution during the winter and break-up in the spring. The length of river covered was 1,820 km, and the winter period ran from October 1 to June 11, 254 days.

The lengths of the open water reaches and the very low temperatures experienced led to very high values of the parameter y, and several cases were found when M reached 0.6, meaning that the floating ice had reduced ice generation by 40%.

Good comparisons between the model results and field observations and surveys for the winters of 1973-74, 1977-78, 1979-80 and 1980-81 were interpreted as indirect proof of the concept as well as establishing the validity of the model.

CONCLUSIONS

An analytical derivation based on simple hypotheses has allowed definition of the role played by floating ice on a river in reducing the surface area active in ice formation, and to propose a Coverage Coefficient which corrects the volume of ice generated over a given period.

The analysis of the phenomenon has led to introduction of the dimensionless parameter y:

$$y = Ka \sqrt{\frac{2}{\rho L k_1}} \sqrt{\frac{Ta\ x}{V}}$$

$$\text{or} \quad y = 1.2 \times 10^{-3} \sqrt{\frac{Ta\ x}{V}}$$

where Ta is the absolute value of the air temperature in °C, x is the length of the open water reach in metres, and V is the surface flow velocity in metres/second.

The Coverage Coefficient then is expressed:

$$M(y) = \frac{1}{y} - \frac{1}{2y^2}(1 - e^{-2y})$$

and the volume of ice produced on the river in a unit time between the 0°C isotherm and a cross-section located at distance x downstream is:

$$P = \frac{B\ Ka\ Ta}{\rho L} M(y)\ x$$

Comparisons with field data showed that the presence of floating ice can reduce the volume of ice generated by as much as 40%, with respect to the traditional calculation procedure, under the most favourable conditions; i.e., long open reaches, low velocities, severe cooling.

NOTATIONS AND UNITS

Notation	Units	Definition	Comments
B	m	Width of the river reach. Assumed constant, may be reduced if shore ice present.	

Notation	Units	Definition	Comments
C		Concentration of floating ice, ratio of the area of the ice pieces to the total area.	$0 \leq C \leq 1$
a		Coefficient $a = \frac{B\ Ka\ Ta}{\rho\ L}$	
b		Coefficient $b = Ka \sqrt{\frac{2\ Ta}{\rho\ L\ ki\ V}}$	
Ka	Kcal/m²/°C/day	Water/air thermal exchange coefficient	Typical Value 460 Kcal/m²/°C/day
ki	Kcal/m²/day/(°C/m)	Thermal transfer coefficient for ice	Typical Value 46 Kcal/m²/day/(°C/m)
h	m	Ice thickness	
L	Kcal/kg	Latent heat of fusion of water	L = 80 Kcal/Kg
P	m³/day	Volume of ice produced during a given time	
Ta	°C	Absolute value of air temperature	
V	m/s	Surface flow velocity	
x	m	Distance measured along the flow	
y		Dimensionless parameter $y = Ka \sqrt{\frac{2\ Ta\ x}{\rho\ L\ ki\ V}}$	
ρ	kg/m³	Specific mass of ice	ρ = 920 Kg/m³

LIST OF REFERENCES

DEVIK (1964) "Present Experience on Ice Problems Connected with the Utilisation of Water Power in Norway" Journal of Hydraulic Research, Vol. 2, No. 1.

HAUSSER AND BEAUCHEMIN (1959) "Rate of Ice Production on an Open Water Surface", AIRH, 8th Congress, Montreal.

MAJEWSKI (1974) "A Study of the Thermal Balance of the St. Lawrence River in Winter Regime", AIRH, River and Ice Symposium, Budapest.

McLACHLAN (1926) Report of the Joint Board of Engineers on the St. Lawrence Waterway Project - Appendix E.

MICHEL, B. (1970) "Winter Regime of Rivers and Lakes" Monograph III - BIA, CREEL, Hanover, N.H.

MICHEL, B. (1978) "Ice Mechanics" Presse de l'Université Laval.

ROBINSON AND WITHERSPOON (1974) "Practical Application of Probability Forecast Water Surface Temperatures of the St. Lawrence River", Seminar on Ice Thermal Regime 1974 - University Laval, Quebec.

WARDLAW (1954) "A Study of Heat Losses from a Water Surface as Related to Winter Navigation", N.R.C.

WEMELSFELDER (1954) "The Influence of an Ice Cover on the Discharge Conditions of a River" Int. Union of Geodesy and Geophysics, Int. Ass. of Scient. Hydr., Vol. 4, pp. 182-190.

TABLE I - HEAT EXCHANGE COEFFICIENTS

Author	Ka BTU/ft²/°F/day	Ka Kcal/m²/°C/day	Location
McLachlan (1926)	95	460	St.Lawrence River
Wardlaw (1954)	100 à 132	484 à 639	St.Lawrence River
Wemelsfelder (1954)	115	557	St.Lawrence River
Hausser and Beauchemin (1959)	82 à 125	397 à 605	St.Lawrence River
Devik (1964)	82 à 125	397 à 605	Norway
Michel (1970)	77	473	New England
Robinson and Whitespoon (1974)	86	416	St.Lawrence River
Majewski (1974) Average December and January	95	460	St.Lawrence River
Average February	75	363	St.Lawrence River

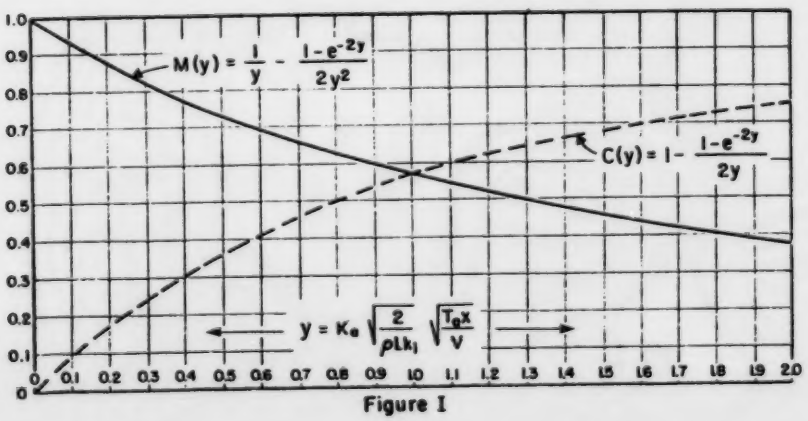


Figure I

DISCUSSION

F. Fonseca

Depending on the turbulence of the flow and the velocity and direction of the wind, a given volume of frazil-ice may cover a widely variable portion of the water surface. Therefore we cannot calculate the surface ice concentration without having first determined the thickness and porosity of the frazil ice layer which is formed on the surface of the flow. Only after having found a way to know the initial thickness and porosity of the surface ice accumulations, would it be possible to calculate the thickening of the ice?

R. Gerard

First, I would like to argue that the simple expression is a classic or traditional approach. I think in a 'long' reach most people try to allow for this coverage effect. The problem comes with trying to decide how thick the slush accumulation (and its porosity) is under the pans. One assumption we have used is that, in a deep river anyway, it depends somewhat on the size of the pans. However, this consideration of slush accumulations under pans seems to be very different from the solid ice cover thickness approach that seems to be used in your paper. Any comments you could make on these different approaches would be appreciated.

Reply

Both these questions have identified the same obvious weak point of our approach. We are perfectly aware of the difficulty of defining slush pan sizes and thicknesses, and the methodology used was consciously developed to skirt the problem. The rivers with which we were dealing at the time did have some slush pans, but they also had more sheet ice formations. The relation was developed to satisfy this sheet ice condition, and when used in the models to check known field data, it yielded good verifications of ice volumes. Without this correction, our models consistently generated too much ice.

As stated in my verbal presentation, we realize that our relation may not be clinically pure and correct, however, it has worked well for us. We would suggest that if others have similar reliable field data and working models of

them, they introduce the coverage coefficient to see if it does not improve the accuracy of the ice volume calculations.

Perhaps their experience will modify the coefficient so we can all have a better relation to use.

SESSION E

MODELLING

Chairman: Dr. B. Pratte; National Research Council of Canada

SESSION E

MODELLAGE

Président: Dr. B. Pratte; Conseil national de recherches Canada

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

COMPUTER SIMULATION OF ICE COVER FORMATION IN THE UPPER ST. LAWRENCE RIVER

Hung Tao Shen[1] and Poojitha D. Yapa[2]

ABSTRACT

A computer model was developed for simulating the formation of ice cover in the Upper St. Lawrence River. The model includes submodels for the river flow condition, the distribution of water temperature or frazil ice production, and the formation of an ice cover. Distributions of water temperature or ice production are determined by a Lagrangian solution of the equation for the transport of thermal energy subject to surface heat exchange. The formation of an ice cover and ice accumulations is formulated according to existing equilibrium ice jam theories. The hydraulic condition in the river system is determined by an implicit numerical solution of unsteady continuity and momentum equations.

[1]U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire; on sabbatical leave from Clarkson University, Potsdam, New York.

[2]Department of Civil and Environmental Engineering, Clarkson University, Potsdam, New York.

SIMULATION DE COMPUTER DE LA FORMATION DE UNE COUVERT DE GLACE DANS LE FLEUVE SAINT-LAURENT SUPÉRIEUR

Hung Tao Shen[1] et Poojitha D. Yapa[2]

RÉSUMÉ

Un modèle de computer été developpé pour simuler la formation de une couvert de glace dans le Fleuve Saint-Laurent Supérieur. Le modéle inclure les sous-modèles pour la condition du fleuve écoulement, la distribution de la température de l'eau ou la production de la glace frazil, et la formation de couvert de glace. Distributions de la température de l'eau ou la production de la glace sont determinées par une solution Lagrangian de la equation pour le transport de la énergie thermique justiciable du échange de la chaleur extérieur. La formation de couvert de glace et accumulations de la glace est formulé conformément aux théories, de la glace emboutiellage equilibre existant. La condition hydraulique dans le reseau de fleuve est determiné par une solution numérique implicite des équations de la continuité chancelant et la force d'impulsion.

INTRODUCTION

Mathematical modeling techniques are increasingly being utilized to analyze river ice problems. River ice phenomena include formation, evolution and breakup of ice covers, hydraulics of flow with or without ice cover, and thermal conditions. All of these interact in such a complex manner that they do not permit closed-form solutions. Utilizing existing theories on constituent phenomena, computer models can be developed to enable engineers to obtain continuous descriptions of the composite river ice process based on a limited amount of field data. In addition, computer models are invaluable for evaluating possible beneficial and detrimental consequences of the design and operation of river control works.

There are a few computer models that can be used to simulate flow and ice conditions in a river (Petryk and Boisvert 1978, Petryk et al. 1981a,b, Simonsen and Carson 1977, and Michel and Drouin 1981). All of these models simulate frazil ice production and the progression, erosion and melting of the river ice cover. Ice cover formation is a major component in these models. Water levels in the river are computed by a backwater procedure. Since the backwater analysis is a steady-state solution, the hydraulic transient effect cannot be accounted for. In a recent study Yapa and Shen (1984) developed an unsteady flow model that is capable of simulating flow and ice conditions in a river network during periods of ice cover. This model requires information on the areal extent of the ice cover. The model thus cannot simulate the period of transition from the open-water condition to the ice-covered condition. A model for simulating the formation of river ice cover is therefore needed. In this paper, results on the preliminary development of such a model for the upper St. Lawrence River are reported. The computer model is programmed in a general manner; it can be applied to rivers other than the St. Lawrence River with appropriate substitutions of physical parameters.

MODEL FORMULATION

River ice cover formation is a complex phenomenon that is affected by many parameters, such as weather, upstream water temperature, discharge, channel geometry, and ice floe characteristics. From the modeling point of view, the formation process can be considered to consist of three interrelated components: the hydraulics of the river flow, the thermal ice regime, and the ice accumulation process.

River Hydraulics

The flow condition in a river not only influences, but also interacts with, all thermal and ice processes. If the effect of phase changes on the conservation of water mass are neglected, stage and discharge conditions in a river with a floating ice cover can be described by the following continuity and the momentum equations:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{1}$$

and

$$\rho \frac{\partial Q}{\partial t} + \rho\left[\frac{2Q}{A}\frac{\partial Q}{\partial x} - \frac{Q^2}{A^2}\frac{\partial A}{\partial x}\right] + \rho g A \frac{\partial H}{\partial x} + (p_i \tau_i + p_b \tau_b) = 0 \tag{2}$$

in which x = distance; t = time; g = gravity; Q = discharge; A = flow area; $H = z_b + d_w + \bar{h}$, water level; z_b = bed elevation; d_w = depth of flow; $\bar{h}$ = equivalent thickness of the ice cover, $\rho_i h/\rho$; h = ice cover thickness; ρ, ρ_i = density of water and ice, respectively; p_b, p_i = wetted perimeter formed by the channel bed and the ice cover, respectively; τ_b, τ_i = shear stress at the channel bottom and at the ice-water interface, respectively. In the present study the numerical scheme developed by Yapa and Shen (1984) is used to solve Equations 1 and 2 for flow conditions.

Equations 1 and 2 show that, in addition to parameters that govern the free surface flow, two parameters can affect the flow condition in a river due to the presence of an ice cover. These two parameters are the reduction in flow area due to the ice cover thickness $\bar{h}$ and the increase in flow resistance due to the additional boundary shear $p_i \tau_i$. Both of these parameters will lead to a change in the water level. For a shallow river with a large ice thickness to flow depth ratio, the effect of ice cover thickness dominates, while in a deep river the boundary shear term dominates.

The estimation of the ice cover geometry, including its thickness and area will be discussed in later sections. The flow resistance term in Equation 2 can be expressed in terms of the commonly used Manning's coefficients (Shen 1983):

$$p_i \tau_i + p_b \tau_b = \frac{\rho g n_b^2 p_b^{4/3}}{A^{7/3}} Q|Q| \quad (1+F)^{4/3} \tag{3}$$

where $F = (n_i/n_b)^{3/2} (p_i/p_b)$; n_i, n_b = Manning's coefficients of ice cover and channel bed, respectivley.

It is known that hydraulic resistance due to the ice cover varies over a wide range throughout the winter. Initially the hydraulic resistance is generally high as a result of the rough bottom surface of the new ice cover formed by the accumulation of ice floes and hanging dams. The resistance coefficient then gradually diminishes during the winter due to the gradual thickening of the ice sheet, the smoothing of the ice surface under the influence of the flowing water, and reductions in the size of hanging dams. The flow resistance reaches a minimum value and increases immediately before the spring break-up with the formation of relief features on the ice-water interface. For an ice cover downstream of an open-water reach, the resistance coefficient may fluctuate during the winter due to the effect of frazil ice transport and accumulation. Based on these observations, a simulation model for the

variation of resistance coefficients of ice covers in the St. Lawrence River was developed (Shen and Yapa 1982, Yapa 1983). Since the geometry of localized ice accumulations such as hanging dams cannot be accurately represented by a one-dimensional form, the resistance coefficient n_i in this model reflects the combined flow resistance effect due to the sheet ice as well as localized accumulations.

The Thermal Ice Regime

River thermal ice processes include the variation of water temperature, the distribution of frazil ice, and the growth and decay of the ice cover. For rivers with negligible thermal stratification, the following convection-diffusion equation can be used to determine the water temperature distributions (Shen and Chiang 1984):

$$\frac{\partial}{\partial t}\left(\rho\, C_p\, A\, T_w\right) + \frac{\partial}{\partial x}\left(Q\, \rho\, C_p\, T_w\right) = \frac{\partial}{\partial x}\left(A\, E_x\, \rho\, C_p \frac{\partial T_w}{\partial x}\right) + B\, \phi \qquad (4)$$

in which T_w = water temperature; B = channel width; C_p = specific heat of water; E_x = longitudinal dispersion coefficient; and ϕ = net heat influx per unit surface area of the river. The net heat influx is composed of heat exchanges through interfaces between the atmosphere, ice cover, river water, and channel bed. In most rivers the dominating component is the heat exchange at the top surface of the water.

For computational efficiency, Equation 4 can be simplified to the following form (Ashton 1982)

$$\frac{\partial T_w}{\partial t} + U \frac{\partial T_w}{\partial x} = -\frac{\phi}{\rho C_p d_w} \qquad (5)$$

in which U = average flow velocity, Q/A. By considering only the heat exchange at the upper surface of the river water, the net heat flux can be estimated by the following simplified relationships:

$$\phi = h_{wa}\,(T_w - T_a) \quad ; \text{without an ice cover}$$

$$\phi = h_{wi}\,(T_w - T_f) \quad ; \text{with an ice cover} \qquad (6)$$

in which h_{wa} = heat exchange coefficient at the air-water interface (22.4 $W \cdot m^{-2} \cdot °C^{-1}$); h_{wi} = turbulent heat transfer coefficient at the ice-water interface ($1622\, U^{0.8} d_w^{-0.2}$ $W \cdot m^{-2} \cdot °C^{-1}$); T_a = air temperature; and T_f = temperature at the ice-water interface (0°C).

Equations 4 or 5 can also be used to determine the distribution of the mean frazil ice concentration in the river (Shen and Chiang 1984). By recognizing that the thermal energy per unit volume of the river water (or water-ice mixture) is either $\rho C_p T_w$ or $-\rho_i L_i C_i$, the frazil ice concentration can be calculated by

$$C_i = - \rho C_p T_w / \rho_i L_i \quad (7)$$

in which L_i = latent heat of fusion of ice and C_i = mean ice concentration in a river cross section.

The thermal growth and decay of a river ice cover can be estimated by finite-difference simulations of the heat conduction in the cover (Ashton 1982, Shen and Chiang 1984, Shen 1984). In the present model, however, a modified freezing degree-day method is used (Shen and Yapa 1983). In this method the ice cover thickness is calculated using the following equation:

$$h = (h_o^2 + \alpha S)^{1/2} - \beta D^{\theta} \quad (8)$$

in which ho = initial ice cover thickness; S = cumulative degree-days of freezing since the formation of the ice cover; D = number of days since the formation of the ice cover; α = empirical coefficient that is a constant during the growth period and decreases linearly with the increase in air temperature during the decay period; β and θ = empirical constants that account for the melting effect of the turbulent heat flux from the river flow. The value of θ is approximately 1.0.

The Ice Accumulation Process

In regions of very low flow velocity, frazil ice forms at the water surface and freezes into a thin cover. In general, however, turbulent mixing will keep frazil crystals entrained in the flow to prevent the formation of a thin static ice cover. Frazil ice crystals in the suspension will agglomerate into flocs and eventually rise to the surface, where they become frozen together to form ice pans or floes. The surface concentration of ice floes increases as they move downstream. This moving layer of the mixture of floes, slush and water may develop into a continuous ice cover, initiated by the formation of an ice bridge at a river section or from an artificial obstacle.

The phenomenon of ice bridging is not well understood, even though under similar flow conditions, ice bridges usually form at the same location each winter. The formation of an ice bridge at a river section is related to the ice transport capacity of the section and the rate of ice discharge coming from upstream. The maximum rate of ice discharge that can pass through a river not forming an ice bridge depends on flow velocity, channel top width between banks or shore ice, surface slope, size of ice floes, and concentration of the ice layer, among other things (Ackermann and Shen 1983, Calkins and Ashton 1975). In the absence of a natural ice bridge, an ice cover can only be initiated at an artificial obstacle such as a dam or ice boom.

Since ice covers in the upper St. Lawrence River are initiated by dams and ice booms, shore ice formation and ice bridging are not considered in the present model. These processes, which may not be very important

in a controlled river such as the St. Lawrence River, are important elements in modeling ice cover formation in uncontrolled rivers and need to be studied.

Once an ice cover is initiated, it will progress upstream through the accumulation of incoming ice floes and slush. By conservation of mass the rate of progression of an ice cover V_{cp} can be calculated as

$$V_{cp} = \left(\frac{Q_i - Q_u}{1-e}\right) / [h_o(1-e_p)B_c] \tag{9}$$

in which Q_i = ice discharge, $Q\ C_i$; Q_u = rate of ice entrained under the cover by flow; e = porosity of individual ice floes; e_p = porosity in the accumulation between ice floes; B_c = width of the newly accumulated cover.

For an ice cover to progress, a stability condition for incoming ice floes must be satisfied. The stability of an ice block can be determined by an equilibrium analysis of the arriving ice floe. This stability criterion was shown to be approximately (Ashton 1974)

$$\frac{U_c}{[gt_i\ (1 - \frac{\rho_i}{\rho})]^{1/2}} = \frac{2\ (1\ \frac{t_i}{H_o})}{[5 - 3\ (1 - \frac{t_i}{H_o})^2]^{1/2}} \tag{10}$$

in which U_c = critical velocity for underturning and submergence; t_i = thickness of the block; H_o = upstream flow depth; and g = gravitational acceleration. For $t_i \ll H_o$, Equation 10 can be reduced to a simpler form (Pariset and Hausser 1961, Pariset et al. 1966):

$$U_c = F\ (\frac{t_i}{\ell_i})\ \sqrt{2g\ (\frac{\rho-\rho_i}{\rho})\ (1-e)t_i} \tag{11}$$

where $F\ (t_i/\ell_i)$ = form factor that varies between 0.66 and 1.30; ℓ_i = length of the ice floe. The effect of porosity e of an ice floe is included in Equation 11. For F = 1.0, t_i = 1 ft, and e = 0.2, one gets U_c = 1.0 fps. Under the criteria given by Equations 10 or 11, an ice cover one floe thick will form by simple juxtaposition. At a higher velocity, ice floes will accumulate at an equilibrium thickness h_o determined by (Michel 1978)

$$\frac{U}{[2\ gh_o(1-e_c)\ (1 - \frac{\rho_i}{\rho})]^{1/2}} = (1 - \frac{h_o}{H_o}). \tag{12}$$

Equation 12 ignores the internal resistance of the fragmented ice cover; therefore, it tends to overestimate the ice cover thickness. From Equation 12 one can obtain a maximum critical Froude number for progression:

$$F_{rc} = \frac{U_c}{\sqrt{gH_o}} = 0.158\ \sqrt{1-e_c} \qquad (13)$$

in which $e_c = e_p + (1-e_p)e$, the overall porosity of the ice accumulation. This equation indicates that e_c has an important effect on the progression of an ice cover. For $e = 0.5$ and $e_p = 0.4$, one gets $F_{rc} = 0.071$. Field observations indicated that F_{rc} can vary from 0.05 to 0.09 for different floe and accumulation characteristics (Kivisild 1959). Newly formed ice floes will have higher porosities and therefore smaller F_{rc} values. When the leading edge of the ice cover reaches a section where F_{rc} is exceeded, incoming ice floes will be carried under the ice cover.

In a wide river the increase in streamwise force may exceed the increase in bank resistance. In this case the internal resistance of the ice accumulation will be unable to resist the increasing streamwise force as the cover progresses upstream. If the stress in the ice accumulation exceeds its internal strength, the cover will collapse and thicken until an equilibrium thickness is reached. This process of mechanical thickening is commonly known as "shoving." Based on the analyses of Pariset and Hausser (1961) and Uzuner and Kennedy (1976), the following equation for the equilibrium thickness h_o can be obtained for steady uniform flows:

$$\left[f_i + \frac{\rho_i}{\rho}\left(f_b + f_i\right)\frac{h_o}{d_w}\right]\frac{U_u^2 B_c}{8g} = \frac{2\tau_c h_o}{\rho g} + \mu(1-e_p)\left(1 - \frac{\rho_i}{\rho}\right)\frac{\rho_i h_o^2}{\rho} \qquad (14)$$

in which f_i, f_b = Darcy-Weisbach friction factors related to the ice cover and the channel bed, respectively; d_w = depth of flow under the ice cover; U_u = velocity under the cover; and B_c = width of the cover. Bank resistance per unit length of the ice cover is quantified as $\tau_c h_o + \mu F_s$, where $\tau_c h_o$ is the cohesive contribution; μF_s is the ice-over-ice friction term; and F_s is the streamwise force per unit width of the cover.

Any ice floe or frazil ice that is swept under the cover will be transported or deposited on the underside of the ice cover. A large accumulation under the ice cover is commonly known as a hanging dam. Because of the lack of understanding of the detail mechanics of undercover transport and hanging dam formation, a simple critical velocity criterion has been used to determine the location and size of hanging dams. Based on observations in the LaGrande River, Michel and Drouin (1975) suggested that the critcal velocity for ice deposition ranges between 0.6 and 1.3 m s^{-1}. Based on field studies in the St. Lawrence River, Shen et al. (1982) found that the shape of hanging dams are affected by three-dimensional flow patterns in the river. Based on the field data they estimated that the critical Froude number for ice cover progression F_{rc} is approximately equal to 0.06.

THE NUMERICAL MODEL

Based on the preceding discussions, the unsteady flow model for rivers with ice covers developed by Yapa (1983) is extended to include the simulation of the formation of an ice cover. The model is developed for and applied to the upper St. Lawrence River to simulate the formation of an ice cover between Iroquois and the Moses-Saunders Power Dam (Fig. 1). In this preliminary study the entire upper St. Lawrence River between the outlet of Lake Ontario and the Moses-Saunders Power Dam is represented by a singly connected channel system consists of 17 segments and 16 nodes, as shown in Figure 2. Channel geometry and other flow-related parameters are summarized in Table I. Flow conditions are determined by an implicit finite-difference scheme, which has been discussed in detail by Yapa (1983). The boundary conditions used are the water level of Lake Ontario at the upstream end and the discharge through the Moses-Saunders Power Dam at the downstream end.

The water temperature distribution is computed by a simple Lagrangian finite-difference procedure, using the recorded water temperature at Kingston as the upstream boundary condition. A Lagrangian method is easy to program and is computationally more efficient and flexible than an Eulerian scheme. This procedure consists of three steps in each time increment of the solution. The first step is to compute the new location of a unit water mass at time $t^{(n+1)}$, which was originally located at a node at the previous time level $t^{(n)}$. The second step is to calculate the water temperature of this water mass at time $t^{(n+1)}$. These two steps should be repeated for all nodes starting from the upstream boundary until the new computed location falls outside the downstream boundary. With the results obtained from the first two steps the new water temperatures at all nodes can be determined by interpolation in the space coordinate. The surface heat exchange term is quantified according to Equation 5. When the computed water temperature is below the freezing point, then frazil ice appears and the concentration C_i can be determined by Equation 7. Once the ice cover is initiated, an additional term account for the amount of ice used for the initial ice cover accumulation and the deposition of ice under an ice cover should be included in the source (or sink) term on the right side of Equation 5.

The ice cover in the upper St. Lawrence River is initiated at the Moses-Saunders Power Dam; its progression upstream is usually accelerated by the flow reduction at the power dam. In the present stage of development, the model uses various stability criteria discussed previously following Equation 9. The computational scheme may be summarized by the flow chart shown in Figure 3.

Sample Simulation Results

Using the computer program, a simulation is made for the ice cover formation during the winter of 1979-80. Figure 4 shows the discharge and air temperature records of this winter. Values of various parameters used in the simulation are summarized in Table II. Figure 5 shows

the water temperature distribution during the formation period. Comparison of the simulated and the observed positions of the leading edge of the ice cover is shown in Figure 6. This figure shows that the computer model can provide good simulation for the progression of the leading edge of the ice cover. Figure 7 shows comparisons of the recorded and simulated head losses. This figure shows that the model is able to simulate the river response through the period of transition from the open-water condition to the ice-covered condition. Figure 8 shows simulated ice cover thickness profiles along the river. Additional refinements in the description of the river geometry and hydraulics within a schematized segment are required in order to be able to give more detailed description of the thickness and shape of hanging dams and the positions of the advancing leading edge between two nodes.

SUMMARY AND DISCUSSIONS

The preliminary development of a computer model for river ice cover formation is reported in this paper. A sample simulation for ice cover formation in the upper St. Lawrence River is presented. Comparison of the simulated results with observed data shows that the model is capable of providing good simulation for the transition period from the open-water condition to the ice-covered condition. The model consists of three interrelated components: unsteady river hydraulics, thermal ice regimes, and ice accumulation processes.

The discharge and water levels are solved from mass conservation and momentum equations using an implicit finite-difference method. The river is represented by a series of single channels connected by nodes. Further improvements, such as representing the river with a network system and other refinements in representing river geometry, are to be made. A Lagrangian method is used in the simulation of the water temperature (or ice concentration) distribution, neglecting the longitudinal mixing effect. The mixing term can be included, but it may not be important except near localized thermal effluents. The ice accumulation process is simulated using existing equilibrium theories of ice jams. Improvements can be made by using more refined theoretical models such as the one developed by Uzuner and Kennedy (1976). The mechanism of deposition, transport and erosion of ice particles underneath an ice cover is not yet well understood; further research is needed on these processes so that the computer simulation can be refined.

ACKNOWLEDGEMENT

This research was partially supported by the New York Sea Grant Institute, through a grant from the National Sea Grant Program, NOAA, U.S. Department of Commerce; and by the St. Lawrence Seaway Development Corporation, U.S. Department of Transportation. A substantial part of this study was done during H.T. Shen's sabbatical leave at the U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, N.H.

REFERENCES

ACKERMANN, N.L., and SHEN, H.T., (1983), Mechanics of surface ice jam formation in river. CRREL Report 83-31, U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, N.H.

ASHTON, G.D., (1974), Froude criterion for ice-block stability. Journal of Glaciology, vol. 13, no. 68, pp. 307-313.

ASHTON, G.D., (1982), Theory of thermal control and prevention of ice in rivers and lakes. Advances in Hydroscience, vol. 13, pp. 131-185.

CALKINS, D.J., and ASHTON, G.D., (1975), Arching of fragmented ice covers. Canadian Journal of Civil Engineering, vol. 2, no. 4, pp. 392-399.

KIVISILD, H.R., (1959), Hydrodynamical analysis of ice floods. 8th IAHR Congress, Paper 23F, Montreal, Canada.

MICHEL, B., (1978), Ice accumulations at freeze-up or break-up. IAHR Symposium on Ice Problems, Lulea, vol. 2, pp. 301-317.

MICHEL, B., and DROUIN, N., (1975), Equilibrium of an underhanging dam at the LaGrande River. Report GCS-75-03-01, Universite d' Laval.

MICHEL, B. and DROUIN, M., (1981), Courbes de remous sous les couverts de glace de la Grande Riviere. Canadian Journal of Civil Engineering, vol. 8, pp. 351-363.

PARISET, R., and HAUSSER, R., (1961), Formation and evolution of ice covers on rivers. ASME-EIC Hydraulics Conference Paper No. 61-EIC-6, Transactions, Engineering Institute of Canada, vol. 5, no. 1, pp. 41-49.

PARISET, E., HAUSSER, R., and GAGNON, A., (1966), Formation of ice covers and ice jams in rivers. Journal of the Hydraulics Division, ASCE, vol. 92, pp. 1-24.

PETRYK, S., and BOISVERT, R., (1978), Simulation of ice conditions in channels. Proceedings of the Specialty Conference on Computer Applications in Hydrotechnical and Municipal Engineering, Toronto, pp. 239-259.

PETRYK, S., PANU, U., and CLEMENT, F., (1981a), Recent improvements in numerical modelling of river ice. International Symposium on Ice, IAHR, Quebec, vol. 1, pp. 426-435.

PETRYK, S., PANU, U., KARTHA, V.C., and CLEMENT, F., (1981b), Numerical modelling and predictability of ice regime in rivers. International Symposium on Ice, IAHR, Quebec, vol. 1, pp. 436-449.

SHEN, H.T., (1983), Hydraulic resistance of river ice. Frontiers in Hydraulic Engineering, ASCE, Cambridge, Ma., pp. 224-229.

SHEN, H.T., (1984), Mathematical modeling of river ice process. Water For Resource Development, ASCE, Coeur d'Alene, Idaho.

SHEN, H.T., VANDEVALK, W.A., BATSON, G.B., and MAYTIN, I.L., (1982), Field investigation of a hanging dam in the St. Lawrence River, winter of 1981-82. Report DTSL55-82-C-C0198A, U.S. Department of Transportation, Washington, D.C.

SHEN, H.T., and CHIANG, L.A. (1984), Simulation of growth and decay of river ice cover. Journal of Hydraulic Engineering, ASCE, vol. 110, no. 7.

SHEN, H.T., and YAPA, P.D., (1982), Simulation of the undersurface roughness coefficient of river ice cover. Report 82-6, Department of Civil and Environmental Engineering, Clarkson College of Technology, Potsdam, N.Y.

SHEN, H.T., and YAPA, P.D., (1983), Simulation of the St. Lawrence River ice cover thickness and break-up. Report 83-1, Department of Civil and Environmental Engineering, Clarkson College of Technology, Potsdam, N.Y. (Also paper accepted for publication in the Canadian Journal of Civil Engineers).

SIMONSEN, C.P.S., and CARSON, R.W., (1977), Ice processes during construction of limestone generating station. Proceedings of the Third International Hydrotechnical Conference, Quebec City.

UZUNER, M.S., and KENNEDY, J.F., (1976), Theoretical model of river ice jams. Journal of the Hydraulics Division, ASCE, vol. 102, no. HY9, pp. 1365-1383.

YAPA, P.D., (1983), Unsteady flow simulation of rivers with an ice cover. Ph.D. Dissertation, Clarkson College of Technology, Potsdam, N.Y.

YAPA, P.D., and SHEN, H.T., (1984), An unsteady flow model of river ice hydraulics. 7th International Symposium on Ice, IAHR, Hamburg.

Table I. Channel Characteristics.

Reach no.	U/S Top width, ft	D/S Top width, ft	U/S Area ft^2	D/S Area ft^2	U/S Datum ft	D/S Datum ft	Length ft	Manning bed n
1	9,999	9,999	900,000	500,000	242.80	242.80	35,000	0.028
2	9,999	9,999	680,000	680,000	242.80	242.80	28,130	0.028
3	20,000	20,000	999,000	999,000	242.80	242.80	27,200	0.025
4	9,999	9,999	501,400	501,400	242.78	242.74	28,100	0.038
5	5,440	5,440	300,000	300,000	242.74	242.56	50,800	0.035
6	7,600	7,600	379,400	142,400	242.56	242.37	75,000	0.030
7	3,240	3,240	142,400	105,000	242.37	242.33	15,000	0.029
8	4,740	4,740	105,000	259,200	242.33	242.21	65,900	0.026
9	3,730	3,760	140,600	80,400	242.20	242.00	17,200	0.036
10	2,570	2,570	89,200	92,900	242.00	240.80	28,100	0.036
11	2,620	2,620	92,900	82,700	240.80	239.90	23,100	0.026
12	2,650	2,650	87,700	108,800	239.70	239.40	16,900	0.031
13	3,528	3,528	108,800	108,000	239.41	239.12	17,300	0.030
14	4,780	4,780	158,700	200,000	239.12	238.17	52,500	0.040
15	4,000	4,000	230,000	230,000	238.17	237.99	20,600	0.040
16	4,500	4,500	240,000	240,000	237.99	237.72	16,000	0.040
17	6,790	6,790	232,500	232,500	237.72	237.50	15,900	0.040

Table II. Summary of parameters used for ice accumulation calculation.

Parameter	Value Used
U_c	1.0 fps
t_i	0.5 ft
e	0.0
e_p	0.4
μ	1.28
τ_c	25 psf
n_i	0.02
n_b	0.026
F_{rc}	0.06
U_{dep}	1.8 fps
U_{ero}	2.0 fps

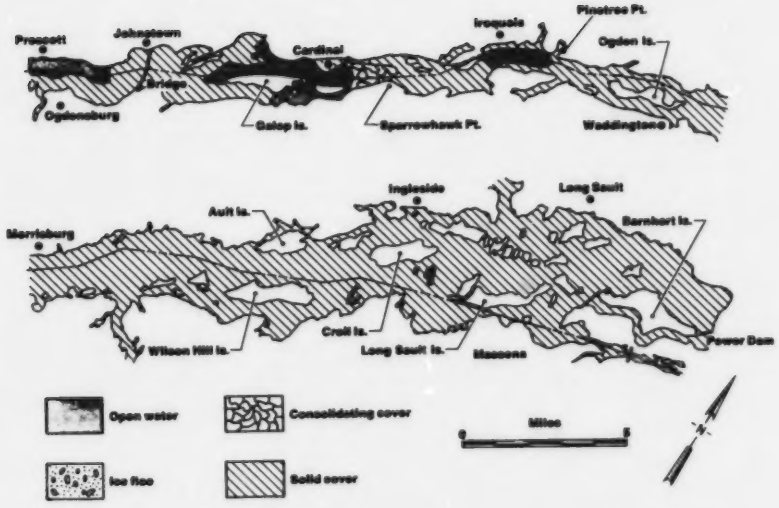


Figure 1. Typical ice condition in St. Lawrence River between Ogdensburg and Massena, New York.

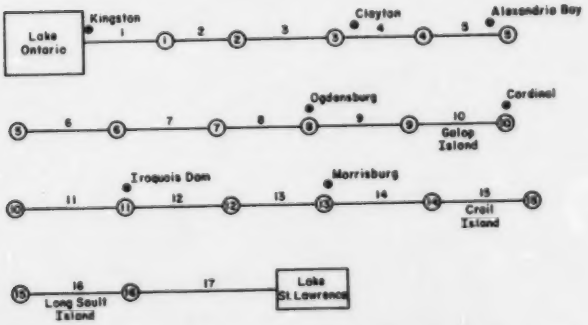


Figure 2. Schematization of St. Lawrence River from Kingston to Power Dam.

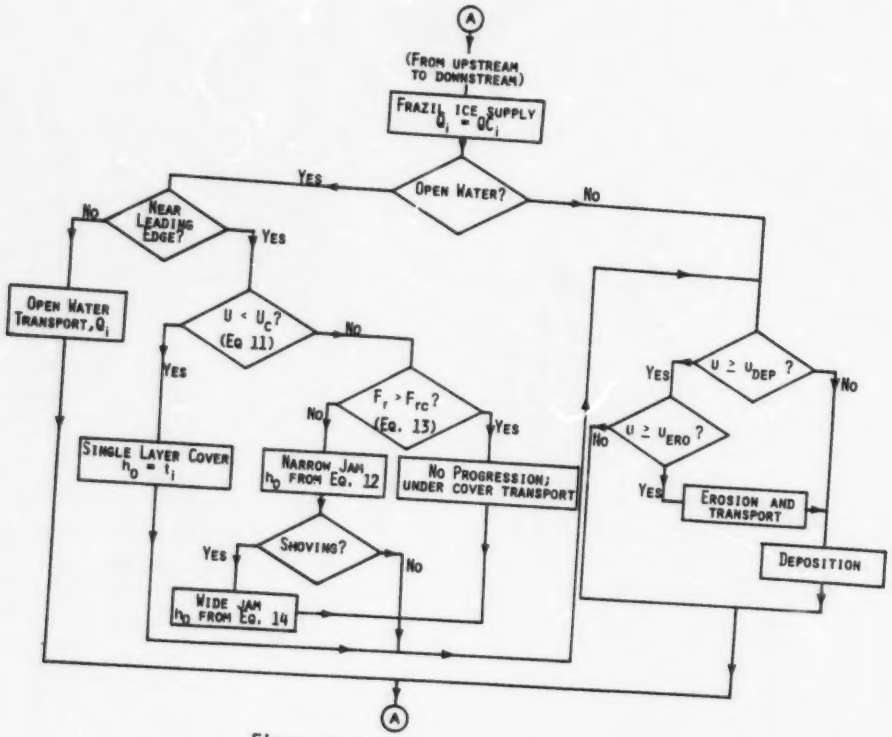


Figure 3. Ice Accumulation Processes

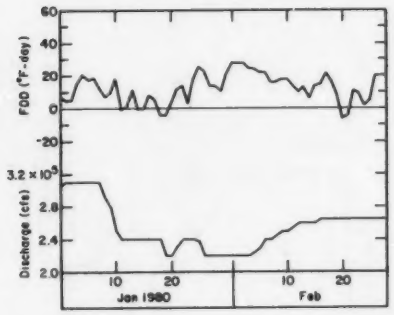


Figure 4. Discharge at the Power Dam and mean daily air temperature in freezing degree-days.

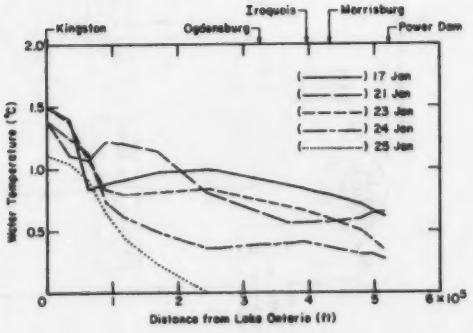


Figure 5. Simulated water temperature distribution along the river, January 1980.

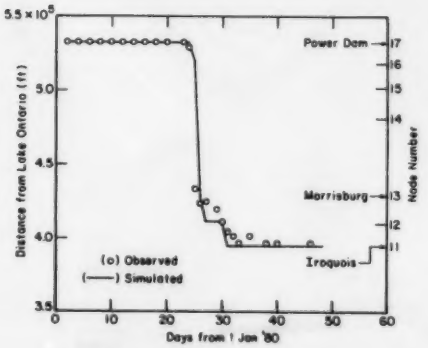


Figure 6. Comparison of simulated and observed ice cover leading edge position.

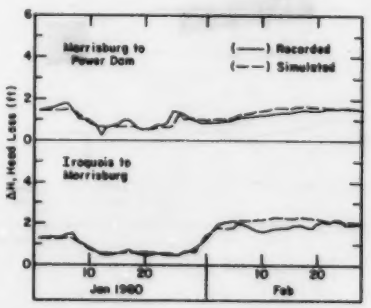


Figure 7. Comparison of simulated and observed head losses.

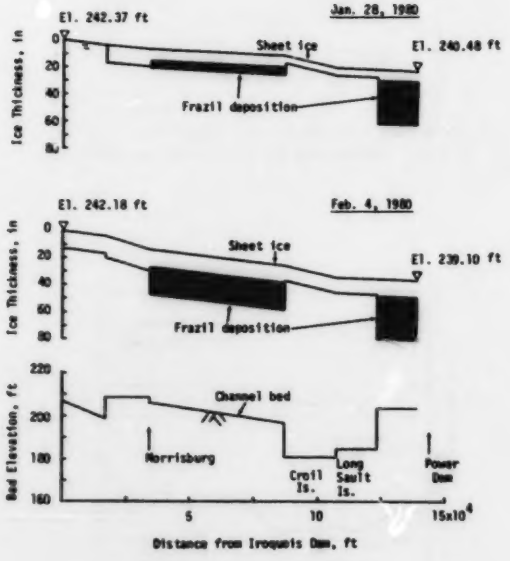


Figure 8. Simulated Ice Cover Thickness Profiles

DISCUSSION

Derek Foulds

There is one year, prior to 1970 when one foot of snow occurred during the ice cover formation period. It would be an interesting test of your model to compare the actual results of ice cover "n" with the computed ones.

Reply

The ice cover resistance model has been compared with the data for the ten winters between 1972 and 1982. It will be of interest to test it with this particular winter suggested by Mr. Foulds.

Robert Gerard

Following on from the previous paper, does your heat transfer algorithm allow for modification due to ice pans? On another point: it is presumed you allow for the 'average' hanging dam thickness explicitly and do not include it in an effective flow resistance. Is this correct? Is any allowance made for shorefast ice growth?

Reply

(1) The computer model can accomodate the coefficient of reduction of ice production. In the present example, however, no such reduction is needed in order to achieve "good" simulation of the ice cover formation. A quick calculation using Mr. Parkinson's curve gives an M value of about 0.75 for the example we presented.

(2) The average hanging dam thickness in each branch is calculated explicitly. This thickness is not used in flow resistance calculation since a one-dimensional description of the hanging dam is not adequate to reflect its effect on flow resistance. The resistance coefficient in the model represents the combined flow resistance effect of the cover and hanging dams.

(3) At the present time, only the formation of the main ice cover is considered. We have not yet analysed the shore ice growth.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU-BRUNSWICK

NUMERICAL SIMULATION OF FREEZE-UP ON THE OTTAUQUECHEE RIVER

D.J. Calkins[1]

ABSTRACT

A numerical model of the flow and ice conditions during freeze-up for the Ottauquechee River has been developed and calibrated with reasonable success. A limited sensitivity analysis of the key ice hydraulic modeling coefficients and independent variables was undertaken to examine their effect on the rate of leading edge progression, ice thicknesses and water levels.

The criteria for advancement of the leading edge were based on both the entrainment velocity of incoming frazil slush at the leading edge and whether or not the flow condition was sub-critical just upstream of the leading edge. The depositional mode of ice thickening accounted for over 50% of the total ice thickness in the steep reaches and over 80% in 1 km of the pool. The simulation suggests that the initial ice cover thickness during progression can be predicted using the equilibrium ice jam theory with a suitable cohesion coefficient.

The inflow ice discharge and ice generated within the reach modeled were important and have to be known with reasonable accuracy to get good simulations of the ice thicknesses, water levels and ice cover progression.

[1]U.S. Army Cold Regions Research and Engineering Laboratory, Hanover, N.H. 03755

SIMULATION NUMÉRIQUE DE LA PRISE DE LA GLACE DANS LA RIVIÈRE OTTAUQUECHEE

par D.J. Calkins[1]

RÉSUMÉ

On a mis au point un modèle numérique de l'écoulement de l'eau et du régime des glaces au cours de la prise de la glace s'appliquant à la rivière Ottauquechee et ce modèle a pu être calibré avec une certaine mesure de succès. Une analyse partielle de sensibilité des coefficients-clés d'hydraulique des glaces entrant dans le modèle et des variables indépendantes a été entreprise afin d'étudier leur effet sur le taux de progression de l'arête d'amont, l'épaisseur de la glace et les niveaux de l'eau.

Les critères de progression de l'arête d'amont ont été fondés sur la vitesse d'entraînement du frasil en bouillie qui descend sur l'arête d'amont et sur la condition d'écoulement, critique ou sub-critique, juste au-dessus de l'arête d'amont. Le mode d'augmentation de la glace par accumulation compte pour plus de 50% de l'épaisseur totale de la glace aux biefs et pour plus de 80% à 1 ku d'un calme. La simulation laisse à entendre qu'il est possible de prédire l'épaisseur de la couche de glace initiale au cours de la progression au moyen de la théorie de l'équilibre des forces d'embâche et du courant avec un coefficient de cohésion approprié.

La quantité de glace d'écoulement et la glace produite à l'intérieur du bief représenté par le modèle revêtent une grande importance et ces données doivent être connues avec passablement de précision pour obtenir une bonne simulation de l'épaisseur de la glace, des niveaux de l'eau et de la progression de la couche de glace.

---

[1] U.S. Army Cold Regions Research and Engineering Laboratory, Hanover (N.H.) 03755

INTRODUCTION

The objective of this study was to simulate the ice/hydraulic processes during freeze-up for a short reach of the Ottauquechee River where extensive field data had been collected. This numerical simulation also examines the impact that the key ice parameters/coefficients have on the "freeze-up process," especially such items as ice thickness, water level and leading edge progression.

It was evident at the beginning of the study that there was no rigorous scientific basis for understanding of the basic mechanisms of some of the ice processes involved. Consequently, some of the equations used to describe these processes are rudimentary and represent only a first order approximation; they may be site-specific for the Ottauquechee River.

Specific goals of the study were to simulate on a daily basis: a) the lateral ice growth rate, b) the ice bridging condition, c) the ice cover thickening and progression rate, and d) the water levels.

PREVIOUS NUMERICAL MODELS

Relatively few published papers have dealt with numerical simulation of the freeze-up ice/hydraulic processes. Among the most recent are Petryk (1981), Marcotte (1981), Petryk et al. (1980), Michel and Drouin (1981), and Shen and Yapa (1984). There are a few models available within the consulting engineering community, but their formulations and logic are not precisely known.

There are two ice/hydraulic models of which this writer is aware that allow one to simulate the effect of an ice cover, one by Potok and Quinn (1979) and the U.S. Army Corps of Engineers HEC-2 program. Keenhan et al.

(1982) reported on simulation of the Peace River freeze-up using HEC-2 with the ice cover option, but this requires the ice cover thickness as user input. The references cited above refer only to those models where the ice processes are simulated on a daily basis in conjunction with the flow hydraulics.

The majority of the models use a gradually varied flow solution for the hydraulic computations; those of Potok and Quinn (1979) and Shen and Yapa (1984) use a one-dimensional unsteady flow simulation. The flow regimes of the rivers to which most of the above models have been applied have always been subcritical. However, the Ottauquechee River passes through critical depth at several sections during freeze-up flow periods.

TEST SITE

The Ottauquechee is located in central Vermont, and the test reach is near the village of Quechee. A complete description of the test site can be found in Calkins and Gooch (1982). The reach is 7.5 km long with run-of-river hydropower plants at each end. The upper dam traps all ice upstream of it, so the ice accumulated between dams is generated only in this 7.5 km reach. No major tributaries enter this section.

The numerically simulated reach is approximately 3 km long. A backwater influence from the downstream dam extends for about 2 km upstream. The remaining 1 km of river has a slope of roughly 0.0015 and a gravel bed of 10- to 15-cm-diameter cobbles. The average channel width is 35–40 m and the mean bankfull depth is 2–3 m.

ICE PROCESSES

The major ice processes active during freeze-up in this reach of the Ottauquechee are 1) instream ice production, 2) lateral ice growth (shore ice), 3) ice cover bridging, 4) thickening and advance of the ice cover, and 5) ice cover melting. The analytical expressions used for each of the processes are described below. They include four modes of ice thickening in addition to the freezing and melting of the ice cover.

Heat Flux at Water Surface

The heat flux Q_w from the water or ice surface to the atmosphere was assumed to be a linear function of the wind speed and the air temperature, such that

$$Q_w = h_{wa} (T_w - T_a) \tag{1}$$

where h_{wa} is the surface heat exchange coefficient, T_w is the water temperature, and T_a is the air temperature. The surface heat exchange coefficient is further defined as

$$h_{wa} = a + bV_a \tag{2}$$

where a and b are constants and V_a is the wind velocity.

Typical values for a and b range from 4-8 (W/m^2-°C) and 2-4 (W/m^3-s-°C) respectively (Dingman et al. 1968, Ashton 1979. This is the only heat exchange component programmed for this model other than inflowing warm water.

Open Water Temperature Decay

The attenuation of the water temperature due to subfreezing air temperatures was simulated using an exponential decay function (Ashton 1979):

$$T_w = \exp\left[\frac{-h_{wa}\,(X-X_o)}{\rho\, C_p\, V\, d}\right](T_{w,o}-T_a) + T_a \qquad (3)$$

where the subscript 0 refers to the upstream cross-sectional value, $X-X_0$ is the distance between cross sections, ρ is the density of water, C_p is the heat capacity of water, V is the open channel flow velocity, and d is the flow depth. The heat transfer coefficient is computed from eqn (2).

Under-Ice Water Temperature Decay

The decay of the water temperature beneath an ice cover is likewise an exponential function except that the heat transfer coefficient for the ice/water interface is calculated differently. In eqn (3) the velocity under the ice cover (V_u) is used, $T_{w,o}$ is replaced by $T_m = 0°C$, and h_{wa} is replaced by h_{wi}. The water/ice heat transfer coefficient (from Calkins 1984) is

$$h_{wi} = \frac{2k_w\, f_i\, R_e\, P_r}{8d\,(1.07 + 12.7\sqrt{f/8})\,(P_r^{.667} - 1)} \qquad (4)$$

where k_w is the thermal conductivity of water, f_i is Darcy's friction coefficient for the ice cover, R_e is the Reynolds number for the entire flow area under the ice cover, and P_r is the Prandtl number. The ice cover melting was calculated based on the water temperature decay beneath the ice cover.

Lateral Ice Growth

The lateral ice growth (shore ice) results from a combination of mechanical and thermal processes. The thermal process is the growth of ice dendrites into the flowing water or the freezing together of ice particles, flocs, floes, etc. The mechanical process is either deposition of ice or abrasion of the dendrites if the ice discharge is too great as it travels past the shore ice. The general form of the equation used for the lateral ice growth in meters per day was

$$L_i = (a_1 V^{b_1}) (T_w - T_a) \qquad (5)$$

where the coefficients a_1 and b_1 were found to vary, depending upon the depth and flow velocity, in order to match the general trend of the observed data. The thickness of the lateral ice cover was assumed to be 0.1 m.

Ice Production

The ice production in the model was calculated by balancing heat loss through the water surface with ice production:

$$Q_i = \frac{h_{wa} A (T_w - T_a)}{\rho_i \lambda} \qquad (6)$$

where Q_i is the ice discharge, A is the open water area generating ice, ρ_i is the density of ice, λ is the heat of fusion for ice, and T_w = 0.0. Again, the surface heat transfer coefficient was computed using eqn (2).

Ice Bridging

One of the goals was to see if a first approximation of ice cover bridging could be established. Critical parameters that could influence ice bridging are the fluid shear stress, bank/floating ice shear stress, ice discharge, and air temperature. A first order approximation assumes the critical width at which the ice would bridge is of the form

$$B_c = a_2 + b_2\tau \qquad (7)$$

where B_c is the critical width, a_2 and b_2 are coefficients, and τ is the flow shear stress. A further constraint was that the ice discharge had to exceed a certain value to meet the above conditions. The flow shear stress was computed as $\tau = \rho gdS$. When the available open water width decreased to a value less than the critical bridging width, it was assumed that an ice bridge formed.

Ice Thickening Modes

There are three main modes of initial ice cover thickening as the cover moves upstream: 1) juxtaposition of incoming ice floes, 2) hydraulic thickening (Michel 1971 or Pariset and Gagnon 1966), which is based on the hydraulics and flow geometry, and 3) an equilibrium ice cover thickness based jointly on the flow hydraulics and mechanics of a fragmented ice cover (Pariset and Gagnon 1966, Uzuner and Kennedy 1976, Beltaos 1983).

1. The juxtaposition mode occurs when the velocity is insufficient to entrain ice beneath the leading edge. In this study juxtaposition is assumed to occur when either the hydraulic or equilibrium thickness criteria calculate the ice cover thickness to be less than 0.1 m. Field observations for the Ottauquechee River indicated this was about the upper limit.

2. The hydraulic thickening equation used to calculate the accumulation thickness t was

$$\frac{V}{\sqrt{2gH}} = \sqrt{(1 - S_i)\frac{t}{H}}\left(1 - \frac{t}{H}\right) \qquad (8)$$

where H is the flow depth upstream of the ice cover, g is the acceleration of gravity and S_i is the ratio of ice to water densities.

3. The equilibrium ice cover thickness equation used is found in Calkins (1983):

$$\frac{B\,V_u^2}{C_*^2\,R_u^2}\left(1 + S_i\frac{t}{R_u}\right) = \frac{2\,c\,t}{\rho_w\,R_u^2} + \mu\,S_i\,(1 - S_i)\,g\left(\frac{t}{R_u}\right)^2 \qquad (9)$$

where V_u is the flow velocity under cover, B is the width of the river at the water line, C_* is the composite dimensionless Chezy coefficient, R_u is the hydraulic radius under the ice cover, c is the cohesion of the accumulating ice cover, and μ is an internal frictional resistance coefficient for the ice cover.

Another important mode of ice cover thickening, once the initial cover has been formed, is deposition beneath the cover caused by entrainment of ice at the leading edge. The entrainment velocity was allowed to be an input variable so that its influence could be evaluated.

The deposition velocity beneath the ice cover was allowed to vary as a function of air temperature. Field observations by the writer on several shallow rivers suggest that as the air temperature decreases, the entrained slush ice tends to deposit at higher flow velocities. Shen (personal communication) has been able to document this phenomenon on the St. Lawrence River. The program presently allows the user to input the critical velocity for deposition. This value is used for air temperatures to -7°C. For air temperature ranges of -7° to -18°, -18° to -30° and -30° to -40°C, the erosion velocity is increased by 1.05, 1.11 and 1.18 respectively. These factors have not been rigorously tested and they

merely represent my experience at this time to account for a process that is difficult to substantiate.

Solid Ice Growth

The solid ice growth of the cover was calculated using the Stefan equation derived and stated by many previous investigators in a daily (i) incremental computation form:

$$\eta_i = \eta_{i-1} + \Delta\eta \tag{10}$$

where

$$\Delta\eta = \frac{1}{\rho_i \lambda} \frac{T_m - T_a}{\frac{\eta_{i-1}}{k_i} + \frac{1}{h_a}} \tag{11}$$

where η_i is the ice thickness, η_{i-1} is the previous day's ice thickness, k_i is the thermal conductivity of ice, and h_a is the heat transfer coefficient over the ice. In the present study, $h_a = h_{wa}$, i.e. the heat exchange coefficient is assumed the same over stationary ice, open water, and a mixture of floating ice in open water.

FLOW HYDRAULICS

A steady-state gradually varied flow backwater program was written to solve for the water surface profile. The standard step method was used. The average conveyance between cross sections was used to determine the friction slope. The HEC-2 backwater formulation was used as the basis for the computational logic. Hydraulics texts describe the standard step method used in this program; it is also described in the HEC-2 Users Manual.

When the lateral ice cover is developing, the program allows for flow to occur under the lateral ice as well as in the open channel. The total conveyance is the sum of the open water and ice-covered flow regions.

The program does not handle overbank flow at this time, nor does it allow for contraction at bridge piers. Expansion and contraction losses for the cross-sectional areas were included, but their contribution is minor for the present case, due to the thick accumulations of ice.

The frictional loss coefficient is calculated using the Manning's equation. The coefficient "n" can vary between sub-reaches but not within the sub-reach. A composite roughness coefficient is computed from the simple but successful relationship (Calkins 1981, Shen and Yapa 1984)

$$n_c = \left| \frac{n_c^{3/2} + n_b^{3/2}}{2} \right|^{2/3} \qquad (12)$$

The test for critical depth at each cross section was determined by checking the computed velocity head $V^2/2g$, if critical conditions occurred at that balanced water surface elevation, and comparing it to one-half the hydraulic depth. If the test ($V^2/2g < 0.95\ A/2T$) indicates critical depth, then the program solves for the minimum specific energy and its corresponding water surface elevation for that section.

MODEL INPUT REQUIREMENTS

The input data consist of the requirements for the hydraulic and thermal portions of the model. The basic data needed to perform the hydraulic backwater computation are cross section geometry, reach lengths, loss coefficients, starting elevation and discharge.

Cross Section

The model accepts irregular-shaped cross sections. Each cross section has its own bed roughness and ice cover roughness coefficient. Additionally, each cross section requires the values for the ice bridging coefficients.

Downstream Boundary

The model needs a daily value for the starting downstream water surface elevation and the daily discharge. A provision exists for decreasing the flow in the upstream direction if a tributary enters the reach.

Upstream Boundary

The upstream boundary inputs are either the inflow ice discharge or water temperature; both values are daily averages.

Meteorological and Heat Flux Coefficients

The air temperature and wind speed are average daily values taken adjacent to the river, or interpolated from the Hanover, New Hampshire, meteorological station if the site measuring equipment was not operating. The surface heat exchange coefficients (a,b) were assumed to be constant for the entire simulation. The lateral ice growth coefficients were initially input as constants for the entire reach, but after several calibration runs the coefficients were treated as functions of flow velocity and depth, which will be discussed later.

Misc Inputs

The cohesion value used for calculations in the equilibrium ice cover thickness equation is constant for all sections and was assumed to be time

independent. The conditions for invoking the hydraulic thickening criterion are based on the ratio $t/H < 0.33$, or the Froude Number $V\sqrt{2gH} < 0.109$. There has been considerable debate as to whether the ratio of $t/H = 0.33$ is the correct value, and a provision in the model allows one to input a value of t/H for the limiting criterion. Setting $t/H = 0$ will default from the hydraulic thickening mode.

MODEL FORMULATION

The computational logic for the ice processes and flow hydraulics was guided by the winter field observations of the Ottauquechee River. Two important considerations were the appropriate time step and the criteria for leading edge advance. The computational technique for all analytical formulations is basically a forward or backward marching procedure, moving either from the upstream or downstream boundary depending upon the process.

Logic

The computational logic for the ice processes and hydraulic computations was the first step in the model development. Figure 1 is a general flow diagram that is used when the inflow water temperature is 0.0°C with or without inflow ice discharge. The individual algorithms for the various ice/hydraulic processes are straightforward but their position in the general computation is important to the success of the calculations. The model must address: 1) When and where will the ice cover bridge across the river? 2) What will be the mode of ice cover thickening along the channel? 3) What are the criteria for the ice cover leading edge advancement?

Ice cover bridging was assumed to occur if a) the critical bridging width calculated from eqn (7) was greater than the available open water width and b) the ice discharge was greater than 0.017 m^3/s (1500 m^3/d). The limiting ice discharge value was based on a combination of field observations, the calibration runs and judgment.

All four modes of ice cover thickening in the model were initially allowed to occur, i.e. juxtaposition, hydraulic thickening, equilibrium thickening and deposition. One of the objectives was to evaluate the different modes of thickening to determine their contribution to the overall thickness. Based on field observations all four modes of thickening were observed, but their contributions were certainly not equal.

The hydraulic thickness is determined by the flow conditions at the leading edge of the cover and this thickness is not altered by the advancing cover. The resulting thickness exists in the field for a short time interval, extending over 1-2 river widths, before the ice cover consolidates to a shorter length and a thicker accumulation. Entrainment of slush ice at the leading edge and subsequent deposition may also occur during the leading edge advance and consolidation steps.

For steep rivers the floe sizes are small due to the continual breakage of the slush ice floes as they travel through the "rapids sections." For the Ottauquechee River, these floe sizes are less than 1% of the river width and may be only 1-2 cm thick or less. As the slush ice travels in a long pool, consolidation occurs by collision and flocculation, increasing the planar dimensions as well as the thickness. The maximum floe size at

the downstream end of the backwater pool is roughly 1 m diameter and the maximum thickness is 6-8 cm.

The telescoping process described earlier is difficult to simulate in the model because the hydraulic conditions can not be easily simulated to the necessary detail, especially when input cross sections are spaced four to five river widths. Reach-averaged quantities for the flow hydraulics were used in the formulations for the ice thickening modes. If the reach conditions indicated a particular thickening mode for ice cover advancement, that initial thickness was not allowed to change; i.e. each initial ice cover thickness was assumed to be its stable thickness and no subsequent shoving of the ice cover was allowed to occur, although deposition was.

The juxtaposition mode of ice cover thickening was allowed to occur if the calculated hydraulic and equilibrium thicknesses were less than 0.10 m. To distinguish between the hydraulic and equilibrium modes of thickening, the upstream hydraulic conditions above the leading edge were evaluated. If $V/(2gH)^{1/2}$ was greater than a certain quantity (initially 0.109), the equilibrium thickening mode was used. Otherwise the hydraulic thickening mode was invoked.

The advance of the leading edge upstream was controlled by two conditions. If a critical depth section is immediately upstream of the leading edge, it must first be submerged to the subcritical state by the water level increases, usually due to downstream ice deposition. The second condition was that the velocity at the leading edge must be less than the extrainment velocity of the arriving slush ice floes. The entrainment

velocity was measured in the field, but it was used as an input variable such that its effect on the leading edge advancement could be evaluated.

Time Steps

The hydraulic computations are made at least daily. In addition, whenever the leading edge reaches a cross section location in a given day, the hydraulic conditions are updated over the entire reach. The various ice processes are simulated on a daily basis.

OTTAUQUECHEE RIVER DATA BASE

The natural river cross sections for this reach were measured every 60 m over the 3000-m length; however, the input cross-sectional shape was a trapezoidal representation for different reaches of the river. Figure 2 shows typical cross sections for the river along with the actual distribution of the ice thickness on 21 December 1981.

The input data of flow discharge, ice inflow, air temperature, and wind speed are given in Table 1 along with additional observed data. The inflow ice discharge in the reach not modeled (= 4.5 km) was calculated using a fixed surface heat exchange coefficient of 17 W/m^2-°C over the open water area upstream. The open area was continually reduced by the percent of lateral ice cover growth.

The average daily air temperature never rose above freezing for the time period 5-22 December, yet the leading edge of the ice cover actually retreated 0.3 km during the 3-day period 11-13 December. The average air temperatures for these 3 days were -2°, -1.5°, and -5°C respectively. The retreat of the leading edge suggests that other heat sources were responsible for melting the ice cover or that the energy budget at the

surface was slightly positive, even though the average air temperatures were slightly below freezing.

The measured water surface profiles during the time frame of 10-21 December 1981 are given in Figure 3. Ice thickness measurements were conducted on 21 December 1981, and the typical conditions are given in Figure 2.

SIMULATION/CALIBRATION

The sequence of model calibration consisted of five steps in the order: open water surface profile, lateral ice growth, ice bridging, leading edge progression and ice-covered water surface profiles.

Open Water

The water surface profiles for open water conditions were first calibrated using 11 trapezoidal cross sections. The average Manning's roughness coefficient was 0.035 for the bed, based on earlier work using the HEC-2 program for the natural sections. During calibration only two cross section shapes and their inverts were slightly adjusted to achieve the desired water elevations for the open water conditions.

The Ottauquechee River is shallow during the freeze-up period. Three sections in the simulation indicated critical depth or supercritical flow and were confirmed by the field observations.

Lateral Ice Growth Rates

The lateral ice growth at all sections was initially assumed to follow the lateral ice growth rate computed for the Nelson River in Manitoba (Calkins and Gooch 1982). However, it was soon very obvious that this relationship was not valid for all reaches of the Ottauquechee River.

After several trial-and-error runs, the coefficients in Table 2 were used in eqn (6).

The coefficients are empirically based and may not apply to other rivers.

Ice Bridging

Once the lateral growth rates were simulated well, the criteria for the initiation of the ice cover to arch at the correct location and at the proper time were examined. Again, taking some data from Newbury's (1968) thesis on measured river widths where bridging occurred in the Nelson River, a relationship was determined such that the coefficients a_2 and b_2 in eqn (9) were 300 and -36.6 respectively.

The first run revealed that these coefficients were not correct. It was decided to keep b_2 constant and vary a_2 to get the best fit. Provision was made for each cross section to have its own set of bridging coefficients. However, when the final calibration was accepted, 85% of the sections had values of a_2 of either 25 or 30.

The other limiting criterion was that in order for an ice bridge to occur, the daily ice discharge had to exceed 0.017 m^3/s. Again, these coefficients and the limiting ice discharge are site-specific.

Leading Edge Progression and Ice-Covered Water Levels

The progression and the corresponding water levels of the leading edge are the key indicators for determining the proper mode of ice thickening. Too fast an advance can mean either that the ice cover thickness is too thin or that the ice discharge is too high. If the ice cover is too thin, then the water levels will be low. If the water levels are correct, and

the ice cover advances too fast, then the ice discharge may be too high. Other variables also play a role in the leading edge advancement and will be discussed later.

The coefficients and input variables in Table 3 were assumed for the first simulation of the leading edge advance and water levels.

The first run produced the following results. The ice cover bridged about 4 days too soon and the leading edge advanced upstream at about the correct rate until day 18, when it stalled and deposition occurred for 2 days. The computed water levels on 20 December were high by about 1 m and the ice cover was also about 1 m too thick. This suggests that the ice discharge was too high.

One of the weak links in the input data was the inflow ice discharge. There were no quantitative measurements. As mentioned earlier, the ice cover actually retreated, even when the average daily air temperatures were below freezing; in other words, other thermal sources were contributing heat, but their quantities were unknown.

Since it appeared that too much ice was reaching the model, the first step was to reduce the inflow ice discharge. The open water surface area was reasonably well documented and needed no adjustment. Since melting of the ice cover had been occurring, the heat transfer coefficient was reduced by 50%. This reduction is also justified by the fact that when floating ice is present on the water surface, the effective heat loss from the river surface area to the atmosphere is reduced.

The resulting inflow ice discharge was reduced by 50% and the coefficient a_2 in eqn (2) was changed from 4 to 2. The resulting simulation of

the leading edge advance was now reasonable and fine tuning could be undertaken.

On the days when the leading edge actually retreated, the simulation indicated an advance because the ice discharge was based on negative daily air temperatures and it indicated ice would be generated. The model could simulate a retreat of the ice cover only if the inflow water temperature was greater than 0.0°C.

On days 12 and 13 the inflow water temperatures were adjusted to 0.04 and 0.07 such that the leading edge would not advance and the ice cover thickness would decrease. If the cross-section spacing had been closer, the melting scheme would have produced a retreat of the leading edge. The positive temperatures were needed because the model still assumed a negative heat loss and would cool the incoming water. In addition, the inflow ice discharge on days 11 and 14 was decreased to 100 m^3/d, to allow for thermal processes not considered that were causing the ice cover leading edge to retreat. The simulation was re-run with these new changes from day 11 to day 14. The leading edge progression then compared very favorably to the field measurements and the water levels on day 20 were within 0.1 m of the field measurements.

The calibration run for the ice cover leading edge versus the actual data is given in Figure 4. This simulation assumed that the hydraulic thickening mode did not occur and the modes of initial ice cover thickness for the advance are juxtaposition and equilibrium thickening. If the hydraulic thickening mode was included in the logic (previous runs), there

was only one reach where it was applicable anyway, otherwise the equilibrium mode occurred in all other reaches.

Figures 5 and 6 show the simulated and measured water surface profiles for the beginning of the ice cover period and 10 days later. The simulation on day 20 is considered acceptable.

Figures 7 and 8 show the computed vs the measured ice cover thicknesses. The comparison is made for two days because it is felt that the simulations using this model are no closer than ± 1 day for the water levels, ice thicknesses or position of the leading edge. The measured ice thickness represents an average of about eight points per cross section. The simulated ice thicknesses are low for day 20 and slightly high for day 21.

DISCUSSION

Sensitivity tests were carried out to examine the influence of the independent variables and coefficients on the leading edge progression, total ice cover thickness and water levels. The ice cover porosity, entrainment velocity and cohesion can all affect the timing of the leading edge by a few days. However, the major effect was the hydraulic constraint of submerging the immediate upstream critical depth section. Additional consideration of this will be forthcoming in a separate report.

To achieve calibration for the ice-covered sections, four additional cross sections were added, just downstream and upstream of the two critical depth sections. Location of cross sections is important. They should be sited just downstream and upstream of critical depth sections, otherwise excessive deposition will be simulated.

Table 4 highlights the initial ice cover thickness and the final thickness achieved on 20 December 1981. The deposition mode of thickening accounts for more than 80% of the accumulation thickness in the "pool section" and nearly 50% in the "uncontrolled" reaches.

CONCLUSIONS

1. This model has demonstrated the importance of the relationships between the various ice processes and flow hydraulics occurring in a shallow river. Typical values of ice/hydraulic coefficients commonly measured in the field were used and gave acceptable results.

2. The role of ice deposition in shallow streams seems to be a dominant feature in the total ice cover thickness, more so than had been previously thought. More in-depth study is needed in this area.

3. The progression of the leading edge is directly related to the entrainment velocity and the occurrence of critical depth. Progression can not occur with a critical flow just upstream of the leading edge.

4. It appears the equilibrium mode of thickening can be used exclusively for the initial ice cover thickness, provided a suitable cohesion value is used which also accounts for the thin frozen layer on the ice surface.

5. The trapezoidal representation of the natural cross section seems adequate, especially in light of the complex ice thickness distribution that is not simulated.

6. The inflow ice discharge to the model is extremely important. Additional work needs to be conducted in this area, especially field observations and measurements on the "effective open water area" and the associated heat flux.

REFERENCES

Ashton, G.D. (1979) Suppression of river ice by thermal effluents. USA Cold Regions Research and Engineering Laboratory, CRREL Report 79-30.

Beltaos, S. (1983) River ice jams, case studies, and applications. Journal of Hydraulics Divison, ASCE, vol. 109, no. 10, pp. 1338-1359.

Calkins, D.J. (1983) Ice jams in shallow rivers with floodplain flow. CJCE, vol. 10, no. 3, pp. 538-548.

Calkins, D.J. (1984) Ice cover melting in a shallow river. CJCE, vol. 11, no. 2, pp. 255-265.

Calkins, D.J. and G. Gooch (1982) Ottauquechee River - Analysis of freeze-up processes. NRC-Canada, Proceedings of the Workshop on Hydraulics of Ice-Covered Rivers, June 1-2, 1982, Edmonton, Alberta.

Dingman, S.L. et al. (1968) The effects of thermal pollution on river ice conditions. Water Resources Research, vol. 4, no. 2, pp. 176-188.

Keenhan, et al. (1982) Analysis of freeze-up jams on the Peace River near Taylor, BC. CJCE, vol. 9, no. 2, pp. 176-188.

Marcotte, N. (1981) Régime thermique et regime des glaces en rivieré: Etude de cas. Proceedings IAHR International Symposium on Ice, Quebec, 1981, vol. 1, pp. 412-422.

Michel, B. (1971) Winter regime of rivers and lakes, CRREL Monograph III-81a, 131 p.

Michel, B. and M. Drouin (1981) Courbes de remous sous les converts de glace de la Grande Riviére. CJCE, vol. 8, no. 1, pp. 351-363.

Newbury, R. (1968) The Nelson River - A review of Subarctic river ice processes. PhD thesis, Johns Hopkins University.

Pariset, E. and R. Gagnon (1966) Formation of ice covers and ice jams in rivers. HYD-ASCE, vol. 92, HY6, pp. 1-24.

Petryk, S. (1981) Numerical modeling and predictability of ice regime in rivers. Proceedings IAHR International Symposium on Ice, Quebec, 1981, vol. 1, pp. 426-436.

Petryk, S. and R. Boisvert (1978) Simulation of ice conditions in channels. Proceedings CSCE Special Conference on Computer Applications in Hydrotechnical and Municipal Engineering.

Petryk, S. et al. (1980) Recent improvements in numerical modeling of river ice. Proceedings of Workshop on Hydraulic Resistance of Ice, Canadian Center for Inland Waters, Burlington, Ontario.

Potok, A.J. and F.H. Quinn (1979) A hydraulic transient model of the upper St. Lawrence River for water resources studies. Water Resources Bulletin, vol. 15, no. 6.

Shen and Yappa (1984) Computer simulation of ice cover formation in the upper St. Lawerence River. Proceedings of 3rd Workshop on the Hydraulics of River Ice, Fredericton, NB, June 20-21, 1984.

Uzuner, M. and J.F. Kennedy (1976) Theoretical model of river ice jams. HYD-ASCE, vol. 102, HY9, pp. 1365-1383.

Table 1.

Day	Flow Discharge m^3/s	Percent[2] Ice Cover	Mean[3] Air Temp °C	Mean Wind[3] Speed m/s	Precip[4] mm	Inflow Ice[5] Discharge m^3/d	Adjusted Ice Discharge m^3/d	Leading Edge 0600-1200 km
1	7.0	-	-5.5	1.0	6	-	-	
2	6.9	-	+.5	1.0		-	-	
3	6.8	-	0.0	0.5		-	-	
4	6.8	-	1.0	0.5		-	-	
5	6.8	-	-1.5	2.0		-	-	
6	6.7	-	-3.0	6.0		-	-	
7	6.7	-	-5.0	2.5	1.5	3628	1814	
8	6.7	-	-6.5	1.5	2.3	4723	2362	
9	6.7	-	-4.5	1.5	1.0	3270	1635	
10	6.7[1]	-	-4.0	1.0		2906	1453	0.6
11	6.6	<5	-2.0	1.0	.3	1453	100	0.88
12	6.6	<5	-1.5	2.0		1090	0	0.67
13	6.5	<5	-5.0	0.5	1.9	3628	0	0.60
14	6.5	<5	-6.5	0	2.1	4723	100	0.67
15	6.4	5	-1.5	0	10.7	1035	518	0.94
16	6.3	5	-1.5	2.0	.1	1035	518	1.34
17	6.3	7	-6.5	0.5	3.0	4394	2197	1.40
18	6.2	8	-6.5	0.5		4245	2123	1.40
19	6.1	10	-12.5	0.5		8175	4088	1.65
20	6.1	15-20	-12.5	0.5		10492	5246	1.76
21	6.1	30	-16.5	0		8393	4197	2.01
22	6.0[1]	40	-7.0	0	11	3052	1526	-

[1]Discharge measurement.
[2]Estimated from oblique photographs.
[3]December 1-8 air temperatures and wind speed records based on Hanover, NH site.
[4]Precipitation amounts measured at Hanover, NH.
[5]Calculated values.

Table 2.

Velocity m/s	Depth m	Coefficients a_1	b_1
> 0.7	> 0.6	.1	-2.81
< 0.7	> 0.6	.4	-.7
< 0.7	< 0.6	.08	-.7

Table 3.

1.	Entrainment velocity	0.6 m/s
2.	Deposition velocity	0.6 m/s
3.	Ice cover porosity	0.5
4.	Hydraulic thickening: t/H value	<0.31
5.	Equilibrium thickening: cohesion	1000 N/m^2
6.	Equilibrium thickening: μ	1.3
7.	Heat transfer coefficients; a,b	4.3
8.	Ice cover roughness: n_i	0.035

Table 4. Comparison of computed initial and final ice thicknesses - 20 December 1981 for run 40-1.

Cross Section	Initial Thickness m	Thickening Mode	Final Thickness m	Deposition Thickness m	% Deposited
1	.1	J	.1	-	
243	.1	J	.28	.18	64
600	.1	J	.55	.45	82
700	.1	J	.89	.79	89
1050	.1	J	1.20	1.10	92
1125	.1	J	1.39	1.29	93
1250	.1	J	.83	.73	88
1463	.84	E	1.01	.17	17
1645	.42	E	.67	.35	52
1768	.10	J	.28	.18	64
2010	.92	F	.92	-	

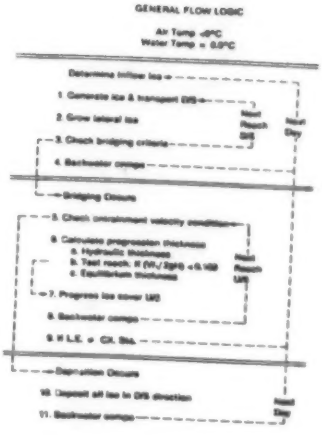


Figure 1. Ice Cover Flow Logic

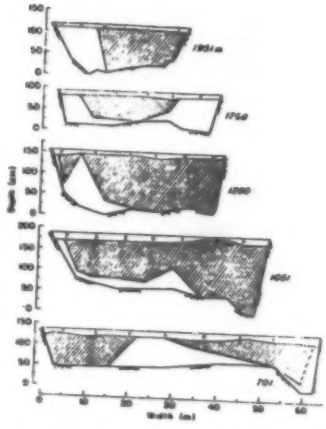


Figure 2. Typical river cross sections and ice thickness distribution

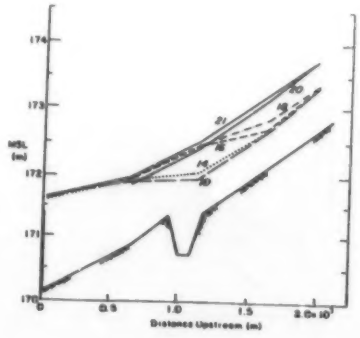


Figure 3. Measured water surface profiles, 10-21 December 1981

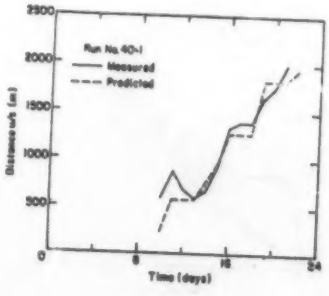


Figure 4. Progression of the loading edge, measured and simulated

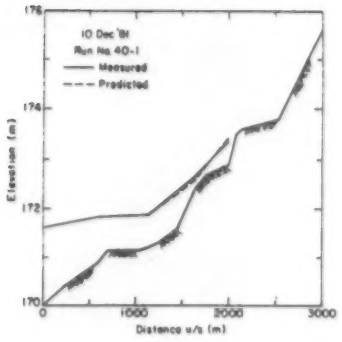


Figure 5. Measured and simulated water surface profiles, 10 Dec 81

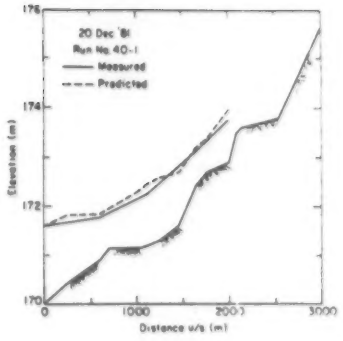


Figure 6. Measured and simulated water surface profiles, 20 Dec 81

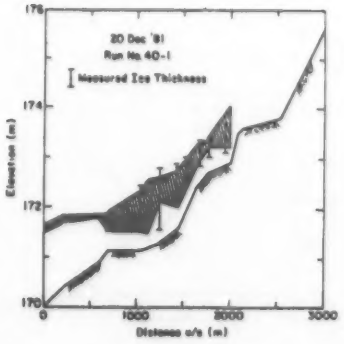


Figure 7. Computed versus measured ice thicknesses, 20 Dec 81

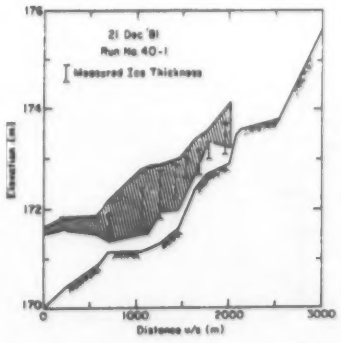


Figure 8. Computed versus measured ice thicknesses, 24 Dec 81

DISCUSSION

B. Michel

We have observed the same processes on a small river bed with a lot of frazil slush. The Froude frontal progression number was 0.012, packing was occurring in the morning at the frontal edge but whatever happened the final characteristics of the cover was that of a hanging dam or your final results are also showing with an approximately constant flow section underneath corresponding to that of a velocity of deposit. I wonder if you could not simplify this complex model for smaller rivers and introduce only this end form of hanging dam formation?

Reply

The model formulation is not really complex but relatively straightforward. The cover progresses upstream according to either juxtaposition, hydraulic thickening or the equilibrium thickness. When the leading edge stalls by encountering either a critical depth section or a flow velocity greater than the entrainment velocity, deposition and the development of the hanging dam begins. I don't believe the hanging dam process could be simplified in the computer simulation.

Robert Gerard

You mentioned that you had to halve the ice discharge you initially calculated given that you had a pretty good handle on the open water area. Do you have any feel for why such a large disparity exists?

Reply

The simplified approach (heat transfer coefficient) for calculating the heat loss from the open water area was inadequate to describe the total energy budget at this time of the year. The contribution of heat from groundwater and tributary flows might have been the major source that was unaccounted and caused the ice cover leading edge to retreat.

James L. Wuebben

On your slide showing the progression of the leading edge, your field data showed a progression and then a retreat of the leading edge. In contrast, your simulation stopped progressing at a point corresponding to the position where the field data eventually retreated. Is this due to the way the model functions, or is it an operator imposed condition?

Reply

The model will calculate a retreat of the leading edge, but the present scheme calls for melting all the ice between the leading edge and the next downstream section before the leading edge is repositioned to that section. I purposely restricted the inflow water temperature such that the ice cover would not completely melt between the sections.

Fernando Fonseca

How do you calculate the shore ice progression?

Reply

The shore ice progression was calculated using equation 5 and the coefficients are given in Table 2 for the various velocities and depths.

S. T. Lavender

COMMENT: The need to deviate from the Newburg function for border ice growth at low velocities on the Ottaquechee River may be a consequence of a change in the mode of border growth. On the Nilrun River (Newburg), the mode of growth is by "buttering". From the author's photos, the mode of growth in the Ottaquechee appears to be "thermal" growth laterally across the water surface.

Reply

The lateral ice growth in the Ottaquechee occurs by both thermal and mechanical processes. In the steep reaches the process is a combination of thermal growth into the deposited anchor ice. The lateral growth in

the pool area was direct thermal growth and freezing together of small ice floes in the low velocity areas of the cross section.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU-BRUNSWICK

WINTER RATING CURVES

and

ICE VOLUME LIMITED WATER LEVELS

S. Thomas Lavender[1]

ABSTRACT

The concept of water level staging during the development of ice covers on steeper, faster flowing rivers to obtain adequately low leading edge Froude numbers to permit the initial formation of a cover has been well studied and understood for some years. In this paper, that concept is expanded to develop two additional useful concepts for the river ice mechanics analyst.

The first concept relates the stage and ice cover thickness achieved during the initial ice cover formation at a site to the stage and discharge relationship that subsequently prevails for the site during, at least, the period of winter immediately following initial formation.

The second concept relates the maximum water level that can be achieved by staging during the initial formation of an ice cover, be it at freeze-up or break-up, to the volume of ice that is available at the time.

[1] Sr. Hydraulic Engineer, Acres Consulting Services Ltd., Niagara Falls, Ontario.

RÉSUMÉ

Courbes du régime d'eau en hiver

et

Limites des niveaux d'eau imposées par le volume de glace

S. Thomas Lavender[1]

Après quelques années d'étude, nous possédons une compréhension à fond du concept de l'étagement des niveaux d'eau pendant le développement des couvertures de glace dans les fleuves d'écoulement rapide, concept qu'on utilise pour arriver à des nombres de Froude pour le bord de la glace en avance suffissamment bas pour permettre la première formation d'une couverture. Dans l'article suivant on trouve une amplification de cette idée, développée pour présenter deux concepts additionnels qui seront utils pour les analystes des propriétés méchaniques des glaces de fleuve.

Le premier concept relie l'étage et l'épaisseur de la couverture de glace réalisée pendant la première formation de la couverture dans un certain endroit au rapport entre étage et débit qui existe subséquemment à cet endroit, au moins pendant la période de temps immédiatement après la première formation de glace.

Le deuxième concept relie la limite du niveau d'eau qui peut se réaliser par étagement pendant la première formation d'une couverture de glace, soit pendant le temps de formation, soit pendant la débâcle, à la volume de glace disponible à ce moment.

[1]Ingénieur hydraulicien principal
Acres Consulting Services Limited, Niagara Falls, Ontario

INTRODUCTION

The concept of water level staging during the development of ice covers on steeper, faster flowing rivers to obtain adequately low leading edge Froude numbers to permit the initial formation of a cover has been well studied and understood for some years. In this paper, that concept is expanded to develop two additional useful concepts.

The first concept relates the stage and ice cover thickness achieved during the initial ice cover formation at a site to the stage and discharge relationship that subsequently prevails for the site during, at least, the period of winter immediately following initial formation.

The second concept relates the maximum water level that can be achieved by staging during the initial formation of an ice cover, be it at freeze-up or break-up, to the volume of ice that is available at the time.

REVIEW OF STAGING PROCESS

Ice covers on steeper sloped and therefore faster flowing rivers can only form by means of the process of juxtaposition. In order for the pieces of ice arriving at the leading edge of the evolving ice cover to be able to remain there, the hydrodynamic forces acting on the pieces of ice must be low enough that the buoyancy of the ice pieces can resist them and not be carried under the already established downstream ice cover. The critical flow condition at which these forces are at equilibrium is conveniently described by the "leading edge Froude number", a well understood and long used concept and criterion (Kivisild 1959; Michel 1971; Uzuner and Kennedy 1972; Uzuner 1977).

Carson and Lavender (1980) presented a method for analyzing field observations of river stage (level) and discharge to define the regime of levels that are possible at a given river section at the time of cover evolution by juxtaposition in the section, whether it be at freeze-up or break-up. The method presents the results of the analysis on a transformed discharge-stage diagram illustrated schematically in Figure 1. The definition sketch in the figure shows the algebraic origin of the variables for the abscissa and the ordinate of the diagram.

In Figure 1, both the open-water rating curve and the regime of stable leading edge conditions are shown. At the time of formation of the cover through a subject reach of river, the water level at the section must lie within the indicated band defined by the range of possible Froude numbers. The stable ice regime in the diagram may be defined by plotting and analysis of observed data as in the noted reference or by calculation from cross-section survey data (Acres 1982).

The actual level that will occur in any particular freeze-up event will depend upon the prevailing properties of the ice at the time; the so-called porosity. The minimum possible level required for juxtaposition to occur is defined by

the higher Froude value of 0.154, while the maximum level occurs for a Froude number of 0.08.

The reduction in hydraulic conveyance of the channel to achieve staging may be effected by deposition of inflowing ice under the established downstream cover up to an "equilibrium" deposition thickness or by a throttling of the channel by a massive ice accumulation (jam) at a channel contraction. The former predominates at freeze-up while the latter tends to occur with an abrupt break-up.

Also shown in Figure 1 is a schematic probability density function for the discharge regime at the time of formation. The broken lines and arrows show the possible water levels - namely, open water, minimum ice staging and maximum ice staging - for the case of the modal (most frequently occurring or the "expected") discharge value. Regulation of the flow by, for example, an upstream reservoir could change the probability of flows at freeze-up. The influence of such flow modification on freeze-up levels is clearly evident from such a diagram.

WINTER RATING CURVES

The control of water levels by the need for leading edge stability as defined by the Froude number is transitory, prevailing only for the time of evolution of the cover through the reach. Once the cover has formed, the flow regime of the river returns to one of uniform flow, with the stage for a given discharge being controlled by the channel friction and cross-section shape. Both these factors are determined in part by the thickness and roughness of the ice cover that has evolved.

The thickness and roughness of the ice cover immediately after formation are a function of the discharge that prevailed at the time of formation. These therefore have an inherent relationship with the stage that was realized at the time of formation. The thickness is in fact determined by the need to have mechanical equilibrium of the ice cover as it evolves. This may be the leading edge equilibrium thickness if the river is "narrow", or the internal equilibrium thickness if the river is "wide" (Pariset, Hausser and Gagnon 1966; Acres 1980a). In either case, the uniform flow equation adequately describes the flow regime for the evolved section geometry and roughness.

With the closure of the river surface by an ice cover, two effects occur to change the stage-discharge relationship for the section. First, the hydraulic radius of the section is halved so that in a wide rectangular channel, for example, the steepness, or slope, of the relationship is increased by about 32 percent. Second, the roughness of the ice cover will act to produce an effective, or composite, roughness co-efficient for the section. Whether the composite roughness is greater than or less than the open water, or bed roughness, depends upon whether the ice cover is rougher than or smoother than the bed. This is apparent from the following formulation for the slope of the winter rating curve expressed in terms of the open water rating curve slope. The expression is derived from the Manning equation for uniform flow (Acres, 1984).

$$(dH_i/dQ) = \{1 + (n_i/n_b)^{3/2}\}^{2/5} (dH_o/dQ) \quad (1)$$

where

H_i = ice covered stage

Q = discharge

n_i = Manning's n-value for ice cover

n_b = Manning's n-value for riverbed

H_o = open-water stage and,

dH/dQ = derivative of stage with respect to discharge (i.e., slope of the rating curve)

For a typical case where $n_i = n_b$, the relationship becomes

$$(dH_i/dQ) = 1.32\ (dH_o/dQ) \quad (2)$$

The slope of the winter rating curve, as can be seen from the equations, will always be steeper than the open water rating curve.

Winter rating curves can be prepared from

-the open water rating curve slope,
-an assumed ratio of ice roughness to bed roughness, and
-the shift in rating curve from the open water level to the cover formation Froude water level.

The upward rating curve shift is determined by the equilibrium ice cover thickness which, as previously noted, is related to the freeze-up staging level as defined in Figure 1. The significance of this is illustrated in Figure 2 for two freeze-up discharge cases. The vertical position of the winter rating curve immediately after freeze-up is fixed by the transitory, Froude criterion defined freeze-up water level. In case 1, the freeze-up level fixing the position of the rating curve is located in the mid-range of possible levels, indicating a moderate value of ice porosity at the time of freeze-up. In case 2, the curve is fixed by a freeze-up level corresponding to a higher discharge and the maximum value of porosity.

In both cases, the shape of the rating curve is similar to the open water curve, but with the slope steepened as indicated by equation (1). Also, in neither case is it necessary to know the actual ice cover thickness or roughness at freeze-up in order to fix the vertical position of the rating curve because this is implicit in the Froude defined freeze-up stage.

There is clearly possible a family of parallel winter rating curves for a particular value of n_i/n_b, with the applicable one in any particular occurrence of cover formation depending upon the discharge and ice porosity at the time of

cover formation. In all cases, once formation has occurred and the vertical position of the rating curve on the stage discharge plane is fixed, the variation in stage with subsequent variation in discharge is defined by the winter rating curve.

Understanding the relationship between the cover formation conditions and the subsequent winter rating curve and being able to even roughly predict that rating curve has potential value for planning ice management in rivers where flows are regulated by upstream storage at, for example, hydroelectric power peaking plants. This concept of winter rating curve determination also can be useful for the interpretation of observed water levels to obtain estimates of stream flows under ice cover conditions. The concept in fact provides a rational basis for one of the three methods of winter streamflow estimation presented by Rosenberg and Pentland (1983).

Before leaving the topic of winter rating curves, two more comments are appropriate, as follows.

First, the rating curves as indicated in the foregoing paragraphs can be expected to apply only for some indefinite period of time after cover formation. The thickness and roughness of a cover will change in time due both to smoothing from heat gain from the water flowing under it and to reductions in discharge which cause settlement and grounding of the cover on shallower bars and verges of natural channels.

Never-the-less, the concept is still potentially useful, particularly if the primary interest in its use is the planning of flow regulating strategies to reduce the risk of incurring ice jam flooding in vulnerable reaches of the river. The effect of such adjustments to the ice cover properties is, on the basis of observations to date, to reduce the stage for a given discharge so that the defined rating curves will always be conservative in this regard.

Second, in the event of a major increase in stage (i.e. increase in discharge), the cover may break up and reform at a new level. This process is no different from, or may well be, a spring break-up event. As in the case of the staging diagram in Figure 1, the rating curve concept is equally applicable in this event. The rating curve as defined by the higher reformation Froude level simply replaces the earlier rating curve.

ICE VOLUME LIMITED STAGING

The process of staging to achieve leading edge Froude stability and the subsequent evolution of the cover through a river reach by juxtaposition requires a certain volume of ice. To illustrate this, the schematic profile of a river reach in Figure 3 has been prepared. The figure shows the state of evolution of a cover at two different times. At the earlier time, the volume of ice accumulated is still inadequate to achieve the critical Froude stage at the upstream controlling section. At the later time, the water level has just reached the critical Froude level and the cover would continue to develop upstream by juxtaposition

with the advent of more ice. Clearly, if the volume of ice available was only equal to that used at the earlier time, the ultimate stage would be limited to a value less than the Froude level.

During the freeze-up period, ice is steadily supplied from ice forming on the open upstream reaches of river and the full volume of ice required to achieve the critical amount of staging is eventually realized. There are circumstances, however, in which the volume of ice may be limited by a short warm winter, an upstream ice cover closure reducing the area of free water surface available to generate ice, or by the influences of upstream storages on the thermal regime of its downstream reaches (Acres 1980b).

In the case of break-up, the volume of ice available is fixed at the "end of winter" volume in place on the upstream reaches of the river. If this volume is limited by the length and width of the upstream reaches or has been depleted by melting and erosion in place prior to actual mechanical break-up, then there may not be an adequately large volume to develop the full, critical Froude depth in the reach for the prevailing break-up discharge. In addition, the break-up discharges tend to be much greater than the freeze-up discharges, even in cases where flow is regulated. Because the volume of ice required to achieve the critical stage for cover evolution increases with discharge, discharge can be a significant faetor in determining whether or not water levels are limited by the available ice volume.

Once the volume of ice available has been consumed and the maximum staging possible with that volume is realized, the water levels immediately upstream of the ice cover will be determined by the uniform flow conditions under the ice cover. Thus, the stage-discharge relationship can again be described by a family of curves on the basic diagram in Figure 1 as before. Such a family is schematically illustrated in Figure 4. In this case, the appropriate curve for any particular occurrence of cover evolution is determined by the volume of ice available. This family of curves is clearly related to the family of curves discussed in connection with Figure 2 because the thickness, and therefore the volume, of the cover when full development (unlimited ice volume) of the cover takes place is a function of the formation discharge.

In the event that a cover of limited volume does not consolidate or freeze and thereby develop strength soon after formation while the discharge continues to increase, the family of curves may be irregular. This would occur as a consequence of the need for mechanical equilibrium of the ice cover at all times. The increasing forces on the cover as a consequence of the increasing discharge may cause a readjustment of the cover thickness (telescoping or shoving) to maintain internal equilibrium. In addition or alternatively, additional downstream under ice deposition may be required as the discharge increases in order to achieve the required staging. Another possibility is the dis-lodgement of the ice accumulation from its current position to a new, downstream lodgement point. Thus, the cover section and therefore the flow cross-section of the channel could change in discrete jumps rather than continuously. Corresponding jumps would be expected in the rating curves.

As in the case of the discharge at formation, the volume of ice available at formation has some probability associated with it. This is indicated conceptually by the ice volume probability density function in Figure 4. The probability of being at any given point in the range of possible positions on the discharge-water surface elevation plane can be determined by combining the probability function for the discharge, carried over in the sketch from Figure 1, with the ice volume function to obtain the combined, or joint, probability of a particular discharge occurring with a particular available ice volume. The joint probability function can be shown as a function of ice volume as indicated in Figure 5.

The concept of a joint probability of discharge and ice volume at the time of an ice cover formation leads to the speculation of a family of iso-risk lines as shown in Figure 5. This in turn leads to the notion of a critical design discharge for a given acceptable level of risk of exceeding a particular critical (flooding) water level. For discharges greater or less than the critical discharge value, water levels will always be less than that for the critical discharge.

CONCLUSION

The foregoing sections have presented some ice mechanics concepts in a gross simplification of the details of the processes, thereby concentrating attention on the essence of the mechanics. While the "real world" is often more complicated and untidy in detail, the principles presented herein are never-the-less applicable. Understanding of these principles is important to the ice mechanics analyst, as he must have a theoretical or conceptual basis for interpreting both observed real world events and the results of computer model analyses. It is hoped that this paper will be of some assistance in this regard.

REFERENCES

Acres Consulting Services Limited (1980a). "Behaviour of Ice Covers Subject to Large Daily Flow and Level Fluctuations." Canadian Electrical Association, Research and Development, Contract G138, December

Acres Consulting Services Limited (1980b). "Dunvegan Power Project, Peace River Ice Study." Energy Resources Conservation Board, Calgary, Alberta, July

Acres Consulting Services Limited (1982). "North Saskatchewan River: Review of Ice Conditions", TransAlta Utilities Limited, Calgary, Alberta, July

Acres Consulting Services Limited (1984). "Behaviour of Ice Covers Subject to Large Daily Flow and Level Fluctuations", Canadian Electrical Association, Research and Development, Contract G272, Vol I, April

Carson, R.K. and S.T. Lavender (1980). "A Discharge/Water Level Relationship for Ice Cover Stability". Proc of Workshop on Hydraulic Resistance of River Ice, Ed. G. Tsang and S. Beltaos, National Water Research Institute, Canada Centre for Inland Waters, Environment Canada, Burlington, Ontario, September

Kivisild, H.R. (1959). "Hanging Ice Dams". Proc 8th Congress IAHR, Vol III, Paper 1-SI

Michel, B. (1971). "Winter Regime of Rivers and Lakes". US Army Cold Regions Research and Engineering Laboratory, Science and Engineering Monograph III-B1a.

Pariset, E., R. Hausser and A. Gagnon (1966). "Formation of Ice Covers and Ice Jams in Rivers". ASCE Journal of the Hydraulics Division, Vol 92, HY6, p 1. November

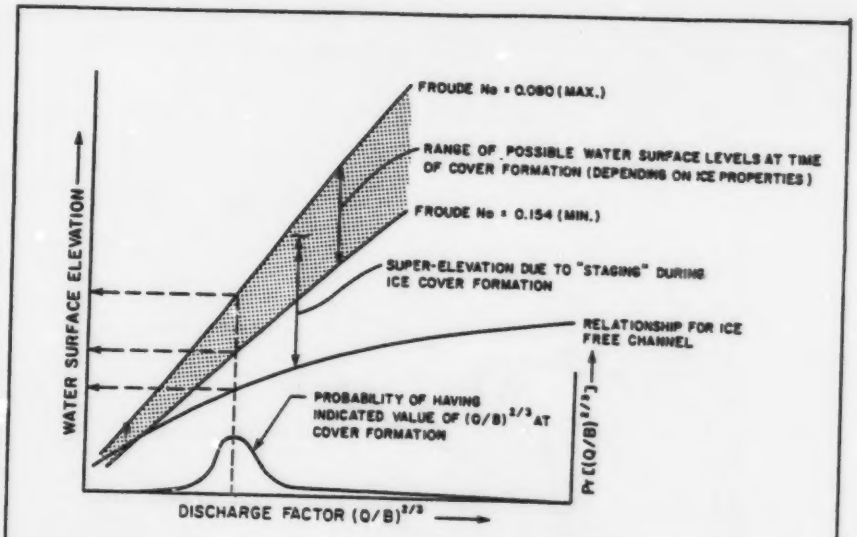
Rosenberg, H.B. and R.L. Pentland (1983). "Accuracy of Streamflow Records", Inland Water Directorate, Ottawa

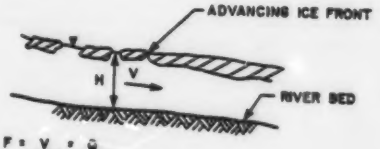
Uzuner, M.S. and J.F. Kennedy (1972). "Stability of Floating Ice Blocks", ASCE Journal of the Hydraulics Division, Vol 98, HY12, p 2117. December

Uzuner, M.S. (1977). "Stability Analysis of Floating and Submerged Ice Floes", ASCE Journal of the Hydraulics Division, Vol 103, HY7, p713. July

NOTATION

B = channel width
d = derivative or differential
F = Froude Number (or FR or Fr)
g = gravitational constant
H = depth of flow
H_i = ice covered stage
H_o = open water stage
n_i = Manning's n-value for ice cover
n_b = Manning's n-value for riverbed
Pr = probability of occurrence of variable(s) in following brackets
Q = discharge
R = risk or probability
V = velocity (Fig. 1) or volume (Fig. 4 & 5)





FROUDE No., $F = \frac{V}{\sqrt{gH}} = \frac{Q}{B\sqrt{gH^3}}$

OR $H = \left(\frac{1}{F^2 g}\right)^{1/3} \left(\frac{Q}{B}\right)^{2/3}$

WHERE $\left(\frac{1}{F^2 g}\right)^{1/3}$ = SLOPE OF LINE

FIG. 1

STAGING DURING ICE COVER FORMATION

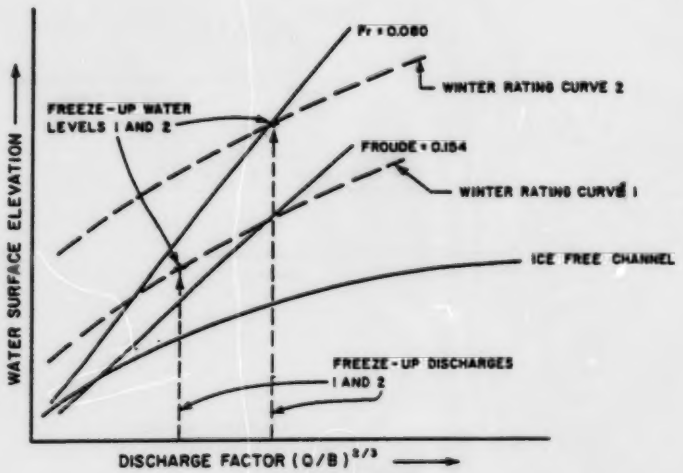


FIG. 2

WINTER RATING CURVES

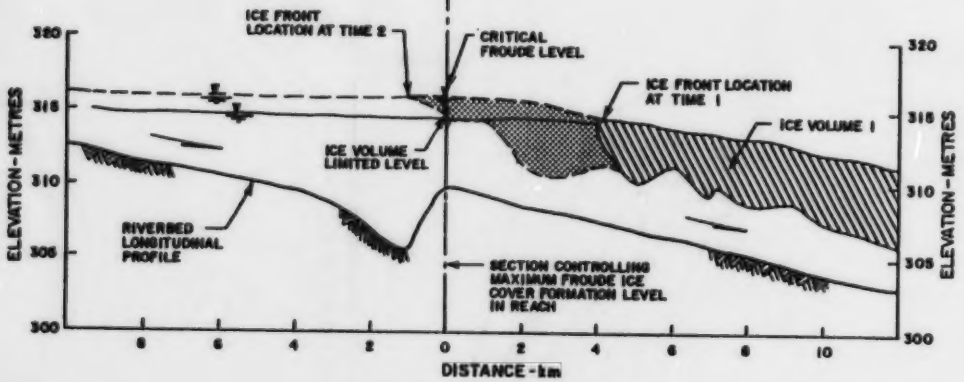


FIG. 3

EVOLUTION OF ICE COVER BY STAGING AND JUXTAPOSITION

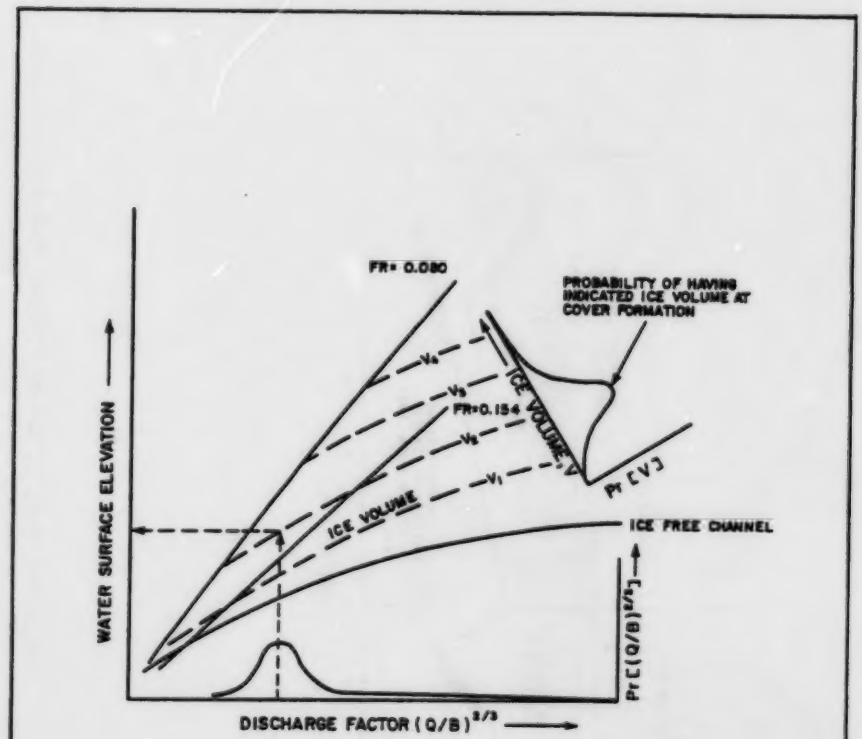
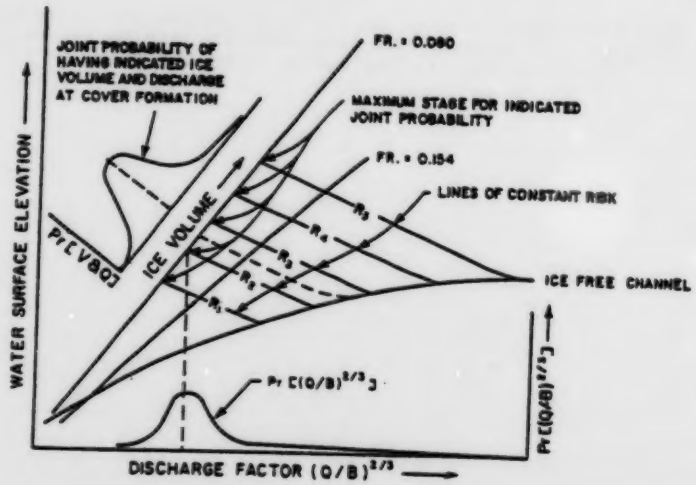


FIG. 4

ICE VOLUME LIMITED STAGING



LEGEND

Q = DISCHARGE
B = CHANNEL WIDTH

FIG. 8

WATER LEVEL-FLOW-RISK RELATIONSHIP

DISCUSSION

Jerry Ford

Have you estimated the confidence intervals for your 25 year envelope?

Reply

NO! The suggested envelope is 'speculative' as indicated in the paper. No attempt has been made to date to estimate or fit a 'joint' probabilities function to the data.

SESSION F

GENERAL

Chairman: Mr. J.G. Lockhart; Saint John River Forecast Centre

SESSION F

GENERALITE

Président: Mr. J.G. Lockhart; Centre de prevision des crues de la riviere Saint-Jean

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21,1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

THE RISE PATTERN AND VELOCITY OF FRAZIL ICE

James L. Wuebben[1]

ABSTRACT

The objective of this study was to examine the rise pattern and velocity of frazil ice. In addition, discs made of other materials were employed both to facilitate this study and to aid in the development of artificial frazil for future transport studies. The rise velocity is a parameter important to the understanding of frazil entrainment, transport and deposition. Laboratory tests were conducted in a large clear plastic cylinder at controlled temperatures. The rise velocity of real frazil is compared with theory and given an indirect verification that the preferential crystal growth direction increases disc diameter while the thickness remains essentially constant. The effective drag coefficients and rise pattern stability are discussed in terms of a Reynolds-Strouhal number relationship. The results from real and artificial frazil experiments are compared, and criteria for frazil simulation are suggested.

[1]Ice Engineering Research Branch, USACRREL, Hanover, NH, USA

VITESSE ET TRAJECTOIRE DE MONTÉE DE LA GLACE FRASIL

James L. Wuebben[1]

RÉSUMÉ

Le but de cette étude fut d'examiner la vitesse et les trajectoires de montée du frasil, paramètres importants pour la compréhension de l'entrainement, du transport et du dépôt du frasil. Pour faciliter cette étude et aider à développer un frasil artificiel, on a aussi utilisé des disques faits de plastic souple. Les essais en laboratoire furent menés dans un cylindre de plastic transparent à des températures contrôllées. Les vitesses de montée du frasil mesurées sont comparées aux prédictions théoriques et permettent de vérifier indirectement que, du fait de l'existence d'une direction préférentielle de croissance des cristaux, le diamètre des disques de frasil augmentent tandis que leur épaisseur demeure pratiquement constante. Les coefficients de traines des disques de frasil et la stabilité de leur trajectoire sont discutés en fonction d'une relation entre les nombres de Reynolds et de Strouhal. La comparaison des resultats d'experience avec frasil réel et frasil artificiel conduit à proposer des critères de simulation du frasil.

[1] Ice Engineering Research Branch, USACRREL, Hanover, NH, USA.

INTRODUCTION

The objective of this study was two-fold. First, the rise velocities of real frazil particles were measured to obtain values for use in studying frazil transport. Second, the feasibility of employing artificial particles for at least some experimental applications was examined.

BACKGROUND

The terminal rise velocity for a buoyant disc will be reached when its buoyancy is in equilibrium with the drag force exerted by the fluid. The buoyant force F_b is equal to the weight of the disc minus the weight of the displaced fluid:

$$F_b = (\rho - \rho_i)\, g\, \frac{\pi D^2}{4}\, t \qquad (1)$$

where ρ_i and ρ are the densities of ice and water, g is the acceleration due to gravity, D is the disc diameter, and t is the disc thickness. Substituting appropriate values yields

$$F_b = 61.64\, D^2\, t \qquad (2)$$

Since the anticipated aspect ratios for the frazil particles are between 1:10 and 1:100, it is reasonable to assume a flat disc geometry. Then, for a circular disc with its plane normal to the direction of motion, the drag coefficient could be calculated as (Albertson, et al. 1960)

$$C_d = \frac{20.4}{Re} = 20.4\, \left(\frac{\nu}{VD}\right) \qquad (3)$$

where the Reynolds number Re is less than 0.5. Here ν is the kinematic viscosity of the fluid and V is the relative velocity of the solid and the fluid. When a disc's plane is oriented parallel to the flow and $Re < 0.1$, the drag coefficient is (Albertson et al. 1960)

$$C_d = \frac{13.6}{Re} \qquad (4)$$

However, the preferred orientation for a freely rising disc is with the plane of the disc normal to the direction of motion. For Reynolds numbers above 10^3, the drag coefficient reaches a limiting value of about 1.12.

As will be shown later, actual measurements of frazil rise velocities extend from the Stokes range to a Reynolds number as high as 20 for a frazil particle 5 mm in diameter.

Examining experimentally derived relations for the drag coefficient of fixed discs such as Figure 1 (see for example Daugherty and

Franzini, 1971), we might expect a drag coefficient ranging from 25 to 2.5 for Reynolds numbers of 1 to 20.

Now, for a flat disc in the preferred orientation (plane of disc normal to flow), we may express the drag force as

$$F_d = C_d \rho \frac{A V^2}{2} \tag{5}$$

where A is the projected area of the disc. When the terminal velocity is reached, we may equate the buoyancy and drag forces such that

$$F_b = F_d \tag{6}$$

$$(\rho - \rho_i) g \frac{\pi D^2}{4} t = C_d \rho \frac{\pi D^2}{4} \frac{U^2}{2} \tag{7}$$

and upon substitution of properties for water and ice near 0°C in centimeter-second units

$$V = 12.53 \sqrt{t/C_d} \tag{8}$$

It is interesting to note that disc diameter does not appear explicitly in equation 8. In the transition region of interest here, however, it does influence the magnitude of the drag coefficient. Since our range of interest lies slightly above the Stokes range, minor adjustments should be made to the simple Stokes relation. However, if we assume that the Stokes result in equation 3 is sufficient for now, substitution into equation 7 yields (in centimeter-second units):

$$V = \frac{(\rho - \rho_i)}{\rho} \frac{g\, t\, D}{\nu\, 10.2} = 431\, tD \tag{9}$$

Actually $C_d \propto Re^{-n}$, where n varies from 1.0 in the Stokes range to zero for fully turbulent flow (Albertson et al. 1960).

EXPERIMENTAL INVESTIGATION

The primary facility for examining the rise pattern and velocity of the particles was a large (30x180 cm) plastic cylinder. Due to visibility problems, a limited amount of data on real frazil was collected in a 1-L graduated cylinder. Time of travel was simply monitored with a stopwatch. Water temperatures were measured but not directly controlled. Data from particles that traveled outside of approximately the center third of the column were discarded. The dimensions of the artificial particles were carefully measured with calipers, and densities were determined by weighing a large sample. For the ice particles, diameters could only be estimated by visual comparison with a scale, and particle thicknesses were not measured. The density of the ice was assumed to be 0.92 g/cm^3.

Real Frazil Data

The data are presented in Figure 2, with a curve bravely sketched in. Given the scatter in the limited available data, there is no strong justification for selecting any particular form of curve; however, the power curve shown

$$V = 0.19\ D^{2/3} \approx 0.2\ D^{2/3} \quad \text{(cm-sec units)} \tag{10}$$

has the desirable property of passing through the origin.

According to the earlier analytical development

$$V = 12.53\ \left(\frac{t}{C_d}\right)^{1/2} \tag{11}$$

so

$$V = 1/C_d^{1/2} \tag{12}$$

Since Figure 1 implies $C_d = 1/D^n$, where n is some exponent between 0 and 1, the experimental results are at least compatible with previous work on fixed discs. It must be remembered, however, that this comparison has not addressed the effect of the frazil having a finite thickness and perhaps a variable aspect ratio.

Gosink and Osterkamp (1982) also studied the rise velocity of frazil in the lab and in the field, and their data are shown in Figure 3. Although their laboratory data are in about the same range as the current study, their fall velocities from the field are far larger. Fitting a power curve to their field data alone would yield

$$V = 4.51\ D^{0.64} \approx 4.5\ D^{2/3} \tag{13}$$

The cause of the disparity in results is not clear but could include an actual difference in crystal geometry or the influence of the differing experimental apparatus. A difference in nucleation and growth conditions could result in differing aspect ratios, which would substantially alter the force balance. In addition, work in the field is more difficult, and they did note some difficulties including interaction of particles due to high concentrations of ice, turbulent eddies and ice formation on their cylinder walls. These conditions are more easily controlled in a laboratory setting. However, either data set is necessarily more correct, since disc thickness was not directly measured in either study.

All particles observed exhibited a nearly stable rise pattern (no rocking or tumbling) unless disturbed by contact with other particles or a boundary. Their orientation was somewhat random, although there

was a tendency for the plane of the disc to be perpendicular to the rise direction.

By assuming that frazil behaves as thin flat discs, we may use the plot of C_d vs Re in Figure 1 to calculate expected rise velocities. By assuming various aspect ratios, 0°C water, and a solid specific gravity of 0.92, we can calculate the buoyancy for a range of particle dimensions. Then, using Figure 1 we can estimate the terminal velocity by equating the buoyancy and drag forces.

Figure 4 presents such an analysis and shows mean values for laboratory data for real frazil. All of the data were generated at roughly the same time and in the same way. Since the measured values cross the plots for various aspect ratios, it appears that t/D is changing.

Due to the expected preferential crystal growth to increase disc diameter, we might assume that disc thickness remains essentially constant. Thus, if the rise velocity at D = 1 mm corresponds in Figure 4 to an aspect ratio of D/t = 0, we could estimate a disc thickness of t_{est} = 0.05 mm. Then if t remains constant as the disc diameter increases, we could directly calculate new aspect ratios for increasing particle size. This is shown in Table I, below along with aspect ratios estimated from Figure 4 for various diameters. In the fourth column, disc thickness was calculated as $t = D_{nom}/(D/t)$.

Thus, within the accuracy of the available data, it appears that the thickness of the real frazil particles was essentially constant for diameters of 1 to 4 mm.

Artificial Frazil Studies

To facilitate the analysis of rise velocity data, discs manufactured from a variety of materials have been used. The artificial discs allowed examination of a wider range of parameters as well as more precise measurements of particle size. However, the real frazil particles have all risen in stable fashion, while some of the artificial particles have oscillated (Fig. 5). It is thought that the oscillations of the artificial discs are related to alternating shedding of vortices. Due to the motion of the particle, the variation in circulation about the particle may also contribute thrust.

These oscillations significantly increase the effective drag coefficient. For a stable rise pattern, a particle rising at or near the wall of a water column is retarded, while an oscillating particle rises faster (even if it repeatedly collides with the wall) since the

frequency and magnitude of oscillation are damped. By substituting data for the artificial particles, we can calculate the effective drag coefficient C_{de} for discs falling unstably using equation 7. Alternatively, the drag coefficient for stable discs can be estimated from relations for a fixed disc such as that shown in Figure 1. In Figure 6 the ratio of effective to fixed-disc drag coefficients vs Reynolds number is shown for polyethylene discs. It is apparent that instabilities due to vortex shedding can substantially increase the drag on a particle.

The drag coefficient vs Reynolds number plot of Figure 1 is redrawn in Figure 7 based on equation 7 and artificial disc data for a variety of material properties and particle geometries. The figure shows an increase in measured drag coefficient relative to the fixed-disc curve beginning at about Re = 100. The data do not extend to high enough Reynolds numbers to detect the upper limit of this effect.

If the oscillations are indeed due to alternate shedding of vortices, we should be able to relate the frequency of oscillations to the Reynolds number by the Strouhal number

$$Sr = \frac{fD}{V} \tag{14}$$

The period of disc oscillation was defined as the time required for a point on the edge of a disc to pass through a complete cycle of rocking above and below the geometric center of the disc. In practice such a point on the edge of the particle was timed through a number of motion cycles starting and ending at its maximum upward excursion relative to the geometric center of the disc.

Figure 8 relates the Strouhal number to the Reynolds number. In addition to data from the current tests, there is also a curve shown for a fixed cylinder transverse to the flow direction, which shows a limiting condition for oscillation frequency. This limit occurs at the onset of complete turbulent mixing. For freely rising (or falling) discs, a limiting condition should also be approached as the discs begin to tumble, but this is beyond the range studied. Much of the apparent scatter in Figure 8 is actually due to the effect of unit mass, ρt, on particle motion. As material density or disc thickness increases, the magnitude and frequency of particle oscillation is damped. This also alters the effective drag coefficients and explains some of the scatter in Figure 7.

According to Figure 8, the oscillation of discs should begin for Re > 150. This suggests that although an ideal fraxil modeling material should duplicate the physical characteristics and dynamics of real fraxil as nearly as possible, artificial fraxil discs with a

terminal Reynolds number below 150 should be adequate for many purposes.

SUMMARY AND CONCLUSIONS

The rise patterns and velocities of both real and artificial frazil discs were investigated in a preliminary attempt to study the transport mechanics of frazil ice. Specific observations include:

1) Data with real frazil of 0.5 to 4 mm diameter yields Reynolds numbers slightly above the Stokes range ($1 < Re < 20$).

2) Scatter in the data makes curve drawing tentative, but to a rough approximation, $V = 0.2\ D^{2/3}$ describes the available rise velocity data. However, this does not address the aspect ratio, which changes as the crystals grow.

3) The aspect ratios of the frazil discs were estimated empirically and found to be approximately $20 \leq D/t \leq 90$.

4) Based on the estimated aspect ratios, the frazil disc thickness remained roughly constant at about 0.05 mm for diameters of 1 to 4 mm. This supports the theory of a strong preferential growth normal to the c-axis.

5) The use of artificial frazil discs was hampered by unexpected unstable rise patterns. The unstable rise pattern increased the drag coefficient to as much as 2.5 times the stable rise value.

6) An evaluation of the characteristics of the oscillations of free discs indicates that the rise pattern should be stable for particle Reynolds numbers less than about 150.

The stability requirement can be conveniently met through the use of polyethylene film commonly used as vapor barriers or trash bags. Surface tension would have to be considered as for all studies employing plastic, but for transport within the water column or under an ice cover this should not be a problem.

Work is now underway to examine the rise velocity of real and artificial frazil flocs, and future work will examine the motion of

discs and flocs in a turbulent flow field. The ultimate aim is an artificial material for use in transport mechanics studies that would simplify the quantification of particle geometry and concentration as well as eliminate the very difficult problem of thermal control.

List of References

Albertson, M.L., J.R. Barton and D.B. Simons, (1960), Fluid Mechanics for Engineers. Prentice-Hall, Inc., Englewood Cliffs, N.J., 561 pp.

Daugherty, R.L. and J.B. Franzini, (1977), Fluid Mechanics with Engineering Applications. McGraw Hill, New York, Seventh Edition, 564 pp.

Gosink, J.P. and T.E. Osterkamp, (1983), Measurements and analysis of velocity profiles and frazil ice crystal rise velocities during periods of frazil ice formation in rivers. Annals of Glaciology, vol. 4, pp. 79-84.

NOTATION

F_b = Buoyancy force (g cm/s^2)
F_d = Drag force (g cm/s^2)
C_d = Drag coefficient
C_{de} = Effective drag coefficient
ρ_i = Solid density (g/cm^3)
ρ = Fluid density (g/cm^3)
g = Acceleration due to gravity (cm/s^2)
D = Diameter (cm)
t = Thickness (cm)
Re = Particle Reynolds number
ν = Kinematic viscosity (cm^2/s)
Sr = Strouhal number
f = Frequency (s^{-1})
V = Velocity (cm/s)
A = Projected area of disc (cm^2)

Table I. Calculated and Estimated Aspect Ratios

Nominal (mm)	D/t t = const	D/t data	t_{est} data (mm)
1	20	20	0.05
2	40	41	0.049
3	60	72	0.042
4	80	88	0.045

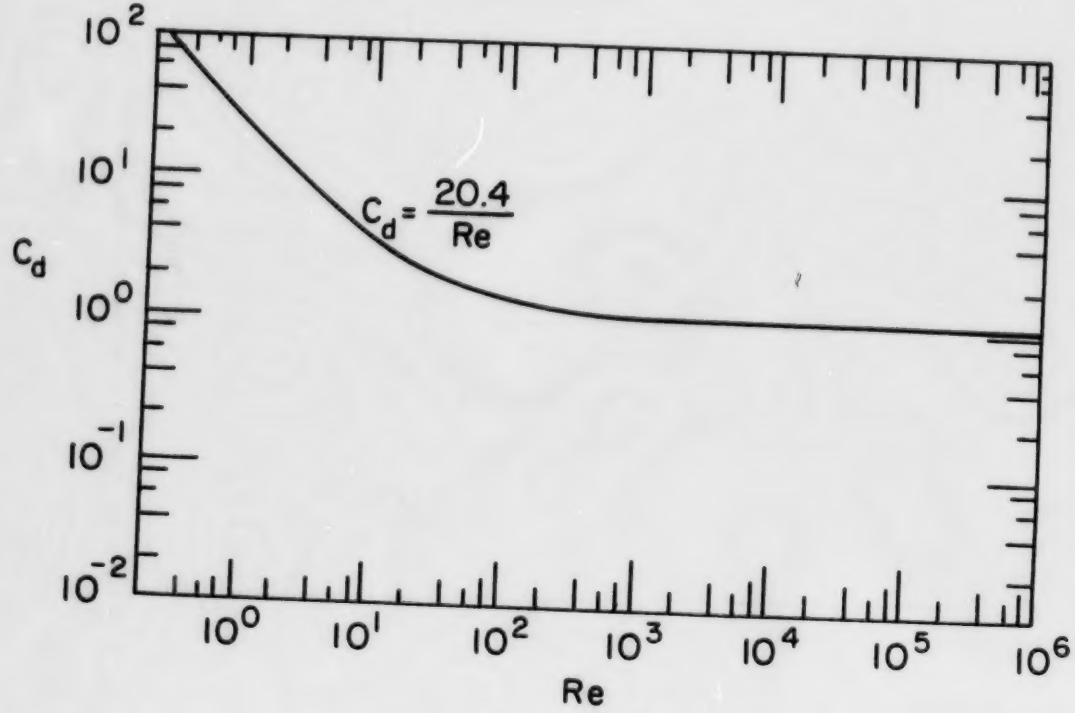


Figure 1. Drag coefficients for fixed discs.

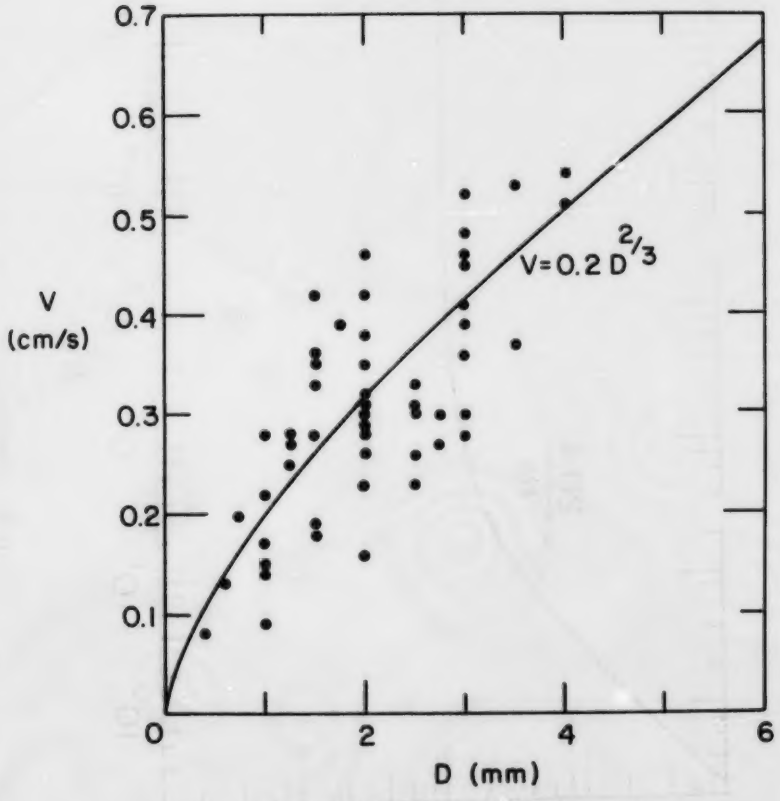


Figure 2. Frazil rise velocity vs disc diameter.

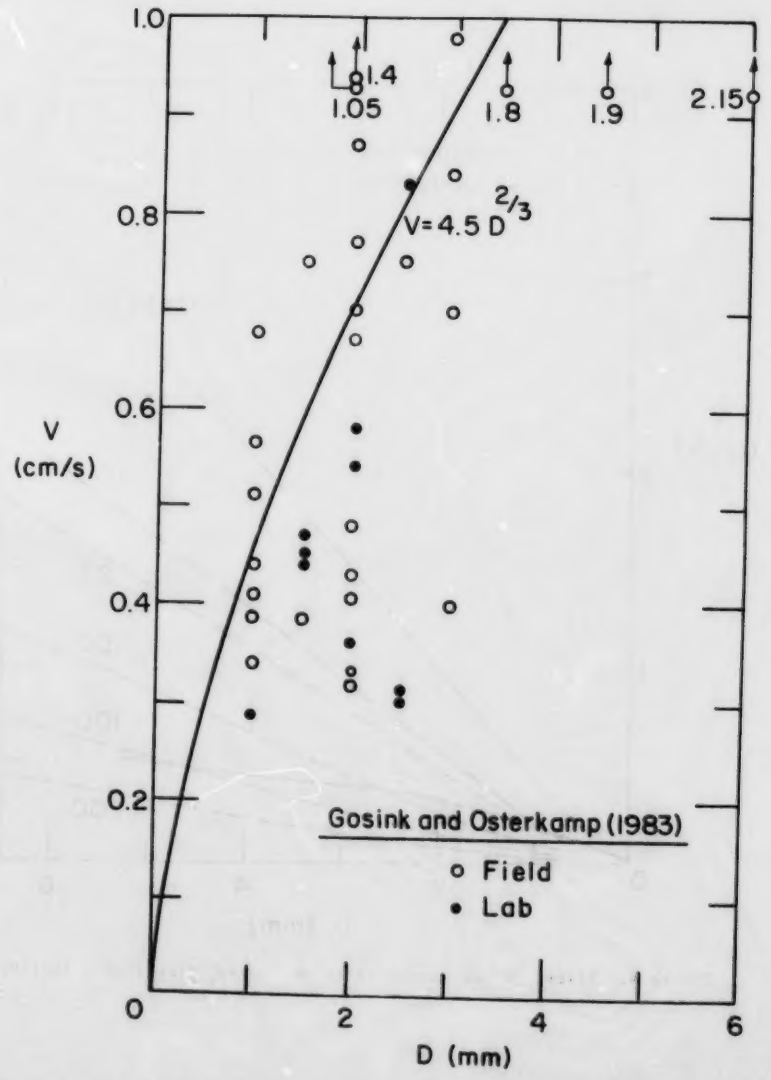


Figure 3. Frazil crystal rise velocity (after Gosink and Osterkamp 1982).

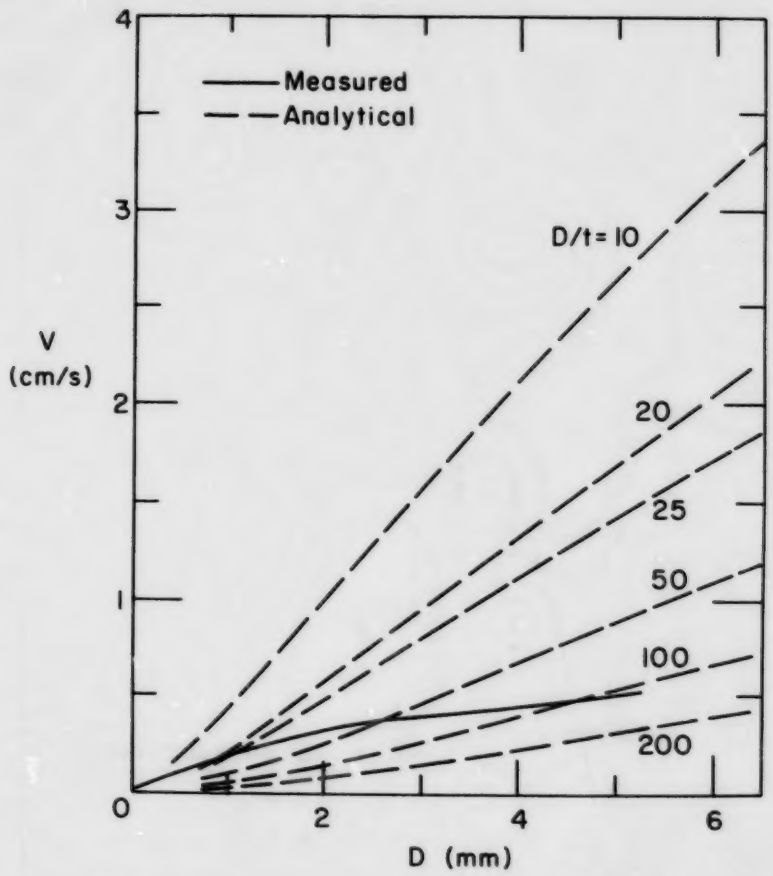


Figure 4. Effect of the aspect ratio on buoyant disc rise velocity.

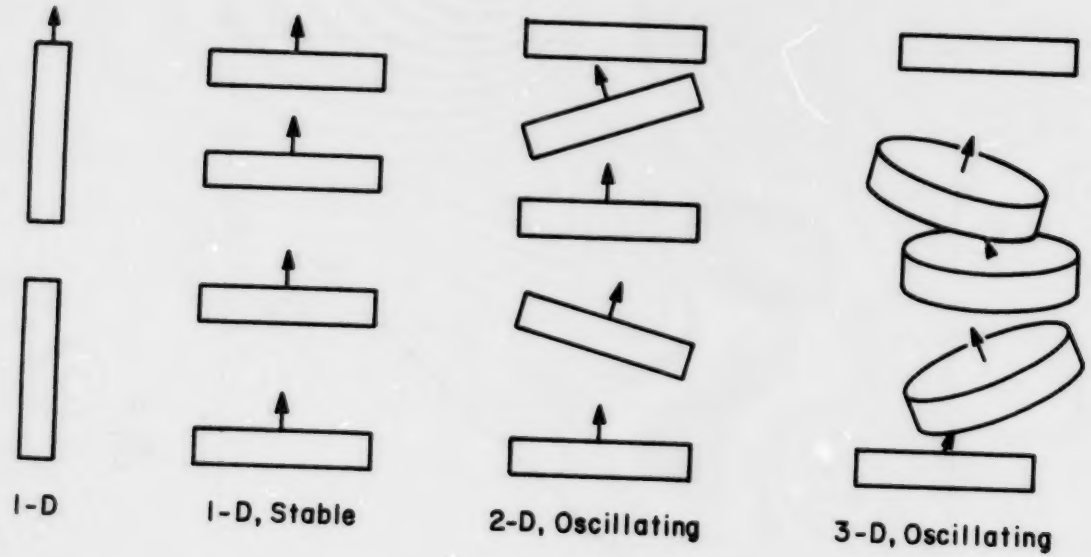


Figure 5. Buoyant disc rise patterns.

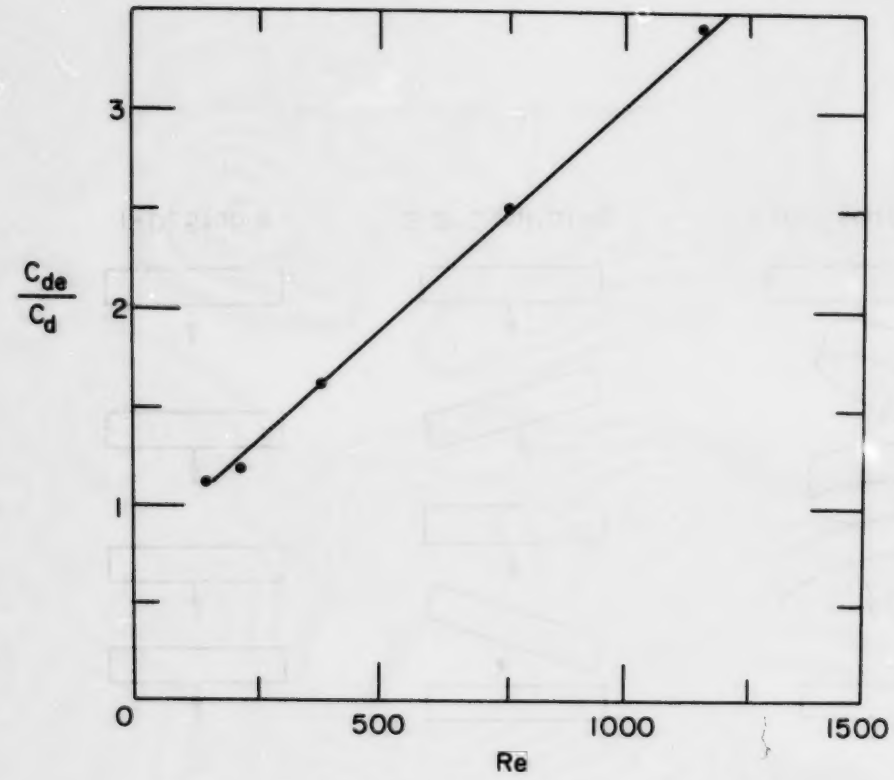


Figure 6. Increase in drag coefficient due to vortex shedding.

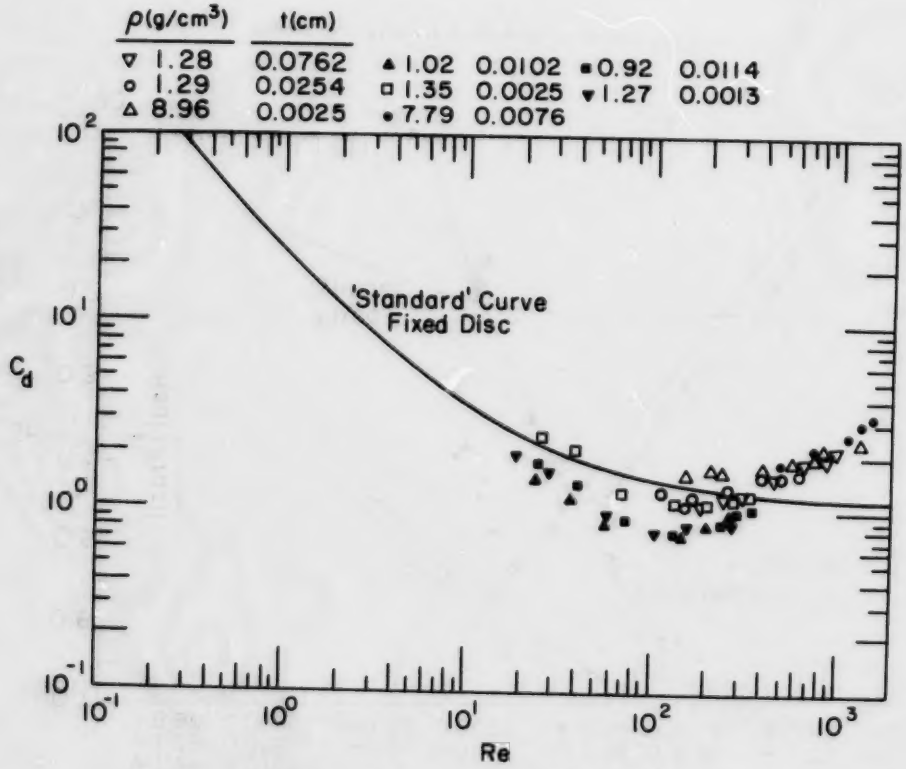


Figure 7. Drag coefficients for free discs.

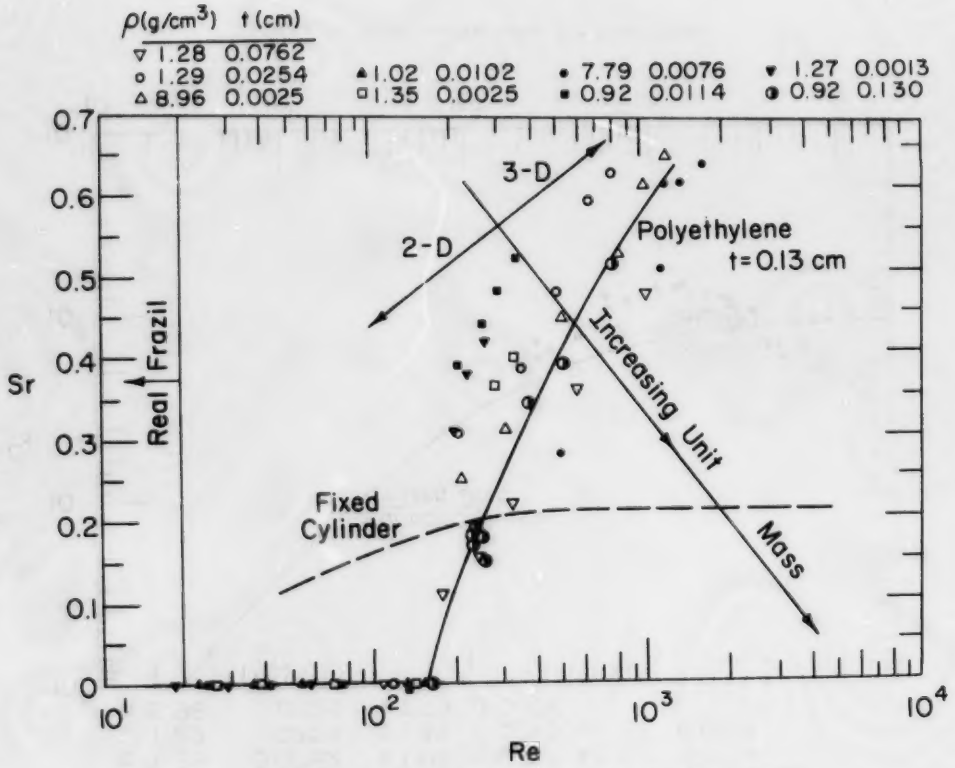


Figure 8. Strouhal-Reynolds relation for free discs.

DISCUSSION

Jerry Ford

Have you studied the effects of viscosity in your experiments?

Reply

I have not tried to directly measure viscosity. However, for the case of low concentrations of individual discs, there is an analagous theoretical development that can be used for illustration. When confronted with the problem of the effect of entrained air on the viscosity of a refrigeration fluid, A. Einstein (1906, Annals of Physics, Leipzig) developed an analytical solution for the case of a low concentrations of solid spheres. His solution may be stated as:

$$\mu' = \mu(1 + 2.5\nu)$$

where: μ' = effective dynamic viscosity of mixture
μ = viscosity of fluid
ν = volume fraction of spheres

If we take the liberty of applying this formula to the case of discs, a 1% concentration of frazil would only result in $\mu'/\mu \approx 1.025$, comparable to a water temperature change of less than 1°C. I would also expect that water containing discreet frazil particles would behave as a dilatant fluid such that the effective viscosity would increase with strain rate. However, for my test conditions and the concentrations of frazil present I feel that the frazil behaved as isolated particles.

K. S. Davar

Comment on effects of frazil on viscosity of water.

Once the water is supercooled and frazil has formed, the frazil particles are likely to coalesce together, deposit on the boundary, and the stickiness of frazil particles would lead to an increased resistance to flow; however, this phenomenon cannot be classified as viscosity.

Reply

Certainly at high concentrations or in the presence of flocculation the problem has more to do with particle interaction and cohesion than with viscosity as normally defined. However, frazil does not necessarily attach to the boundary and become a rigid surface. I have sampled massive deposits of passive frazil ice consisting mainly of individual discs on the Tanana River in Alaska. This frazil was experiencing active transport. There were also "eggs" of ice that were clearly composed of frazil discs that had previously flocculated. Apparently these flocs continued to be transported, compressed and abraded until rounded pieces of solid ice resulted. Individual discs could still be identified within these ice pieces that resembled well-rounded stones within a river gravel. Again, the situation might not be defineable in terms of viscosity, but it is possible to have cohesionless transport of frazil particles.

R. Gerard

Could it be concluded from your results that the rise velocity of frazil discs can be predicted using the standard drag coefficients for discs if the disc aspect ratio is known?

Reply

It depends on the accuracy desired. If there was a practical method to measure or estimate the aspect ratio of a frazil disc, I'm sure that a force balance using a standard drag coefficient relationship would work at least as well as direct measurement of rise velocity. That would also remove effects due to experimental apparatus or particle interaction and yield a "purer" number. In addition, until it is possible to measure or predict the aspect ratio and growth characteristics, it will be difficult to rationalize the differing results such as those of the present paper and the 1982 paper by Gosink and Osterkamp.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

MEASUREMENTS OF FRAZIL AND DRIFTING ICE IN THE LACHINE RAPIDS

B. Michel[1], P. Desroches[2],

J.-G. Dussault[2], F. Fonseca[2] and D. Levan[2]

ABSTRACT

This paper presents part of the results of a field survey made by Hydro Québec at the head of the Lachine rapids during the winter 1982-83 to study the characteristics of the in order to evaluate the feasability of the hydroelectric option in the "Archipel" project.

Contrary to expectations, the observations have shown that most of the drifting ice which is observed during day time is not frazil, but drifting pellicular ice formed the preceeding night in lake St. Louis. This condition of ice formation has been evaluated for the first time.

Frazil was formed at night only for a limited period of time in February 1983, because of an exceptionnaly warm winter. It is formed in water supercooled only about 0,01°C, during the phase of residual supercooling. This frazil is active and produces deposits on targets used for its detection. The characteristics of these deposits are discussed as well as the mechanisms involved. A law of growth of anchor ice is proposed.

[1] Recherches B.C. Michel, Inc.

[2] Service Hydraulique, Hydro Québec

RÉSUMÉ

Cet article présente une partie des résultats de la campagne de mesures effectuée par Hydro Québec au cours de l'hiver 1982-83, dans la partie amont des rapides de Lachine, en vue d'évaluer la faisabilité de l'option hydro-électrique du projet "Archipel".

Contrairement à ce que l'on attendait, les observations montrent que presque toute la glace de dérive qui est observée le jour à cet endroit est de la glace pelliculaire en mouvement, formée la nuit précédente dans le lac St-Louis. Il a été possible de préciser, pour la première fois, les conditions de formation de cette glace.

Le frasil s'est formé la nuit seulement au cours d'une période de temps limitée à février 1983, en raison d'un hiver exceptionnellement doux. Il se forme essentiellement dans de l'eau surrefroidie d'environ 0,01°C seulement, durant la phase du surrefroidissement résiduel. Ce frasil est actif et produit des dépôts sur diverses cibles utilisées pour sa détection. Les caractéristiques de ces dépôts sont discutées ainsi que les mécanismes de leur formation. On propose une loi de croissance de la glace de fond.

INTRODUCTION

The "Archipel" project in Québec is a comprehensive water enhancement project for the Montréal archipelago, shown on Figure 1. The main aspects of this project are flood control in three connecting rivers, water pollution abatement of all surrounding water bodies and enhancement of shores and beaches for esthetic and recreational purposes.

One major aspect of this overall scheme is the derivation of flood waters from heavily flooded areas on the shores of lake of Two Mountains, Rivière des Milles Iles and Rivière des Prairies, to the St. Lawrence River. This would require that some structure be built at the head of the Lachine rapids to control the water levels and prevent flooding of the heavily populated areas along lake St. Louis, during the water diversion periods.

The construction of a flood control structure at the head of the Lachine rapids had naturally lead Hydro Québec to consider the feasability of combining this structure with a dam for hydro power production.

Studies of hydro power schemes in the Lachine rapids had been done before by Hydro Québec. These studies have lead, including this more recent one, to the conclusion that the cost of building a power plant which would operate in winter by the standard practice of maintaining a stable ice cover in front of the dam, was too high for economical production of power. This is due to huge costs needed for excavations in the rapids, in order to reduce the water velocities to acceptable levels. Otherwise ice jamming with thick ice covers might possibly flood Montréal Island.

The only solution left to lower the cost of construction is to operate the power plant, in the winter months, without an ice cover in the forebay. This solution, however, presents unsolved technical problems that had to be dealt with. No power plant has yet been built in the world that would have to pass active frazil in the turbines during the winter months on a regular basis and a few small plants are operating in a satisfactory manner by evacuating continuously the floating drifting ice. Hydro Québec thus undertook, during the winter 1982-83, extensive field measurements to deal with these problems. We will discuss here, only the major field surveys done at the future construction site.

DESCRIPTION OF POWER PROJECT

A general layout of the project is shown on Figures 2 and 3. The axis of the dam is about 1,5 km downstream from the Mercier bridge at the head of the Lachine rapids. A boom in front of Dorval island would stabilize the solid ice cover in lake St. Louis in lower velocities areas. Downstream from this structure there would be open water flow in a reach, approximately 5 km long, leading to the dam.

The major works consist of a diversion canal on the south shore that could by-pass 7000 $m^3 s^{-1}$ during flood time. This canal is combined with the tailrace canal of the power station downstream from the dam.

At the dam site there is a wide multi-purposes regulating spillway that will be used to maintain a minimum flow in the Lachine rapids at all times, pass floating ice in winter and, evacuate part of the floods at spring time.

At the southern end of the spillway the power plant is inclined relative to the flow direction and may be equipped with either 6, 12 or 16 bulbs units for a total capacity of 225, 415 MW at load factors close to 100% or 455 MW. The head is between 9 and 10 m depending on modes of operation.

A special interconnecting opening in the diversion canal has been provided, to increase the surface flow to the spillway during the winter months, and improve the evacuation of drifting ice floes.

OBJECTIVES OF THE FIELD SURVEYS

The specific objectives of the field surveys, at the site of the project, were to determine the characteristics of the ice that would have to pass through the turbines and over the ice spillway. Two types of ice formation had to be considered:

- characteristics of drifting floating ice to be passed over the spillway (discharge, thickness, geometry, strength, etc.);
- adhesivity of active frazil and volume of deposits on various objects.

This program was carried out by daily aerial surveys, measurements from the shore and at a special measuring station set under the lower trusses of Mercier bridge, more specifically built to study the properties and effects of frazil.

CHARACTERISTICS OF DRIFTING ICE

Ice types

The main observations were taken by oblique pictures from Mercier bridge during daylight hours. These were completed by aerial photographs taken by helicopter.

The types of drifting ice observed were: frazil disks, frazil slush, ice plates and drifting anchor ice. These various types of ice are shown on Figures 4 and 5.

The individual frazil disks had dimensions up to a few cm of average diameter. They were observed in depth with frazil collectors. Their concentration was relatively high in the upper meter of flow.

Frazil slush was seen in all cold weathers, mixed with other ice types. There were, however, very few classical ice pans because frazil had little distance to evolve in the short drifting reach since its formation.

Ice plates were the dominant drifting ice type during daylight hours, for all weather conditions at the head of the rapids, under Mercier bridge. They are formed in low velocity areas on the north side of lake St. Louis (Dorval Bay) and also in the main channel of the river in the lake. Very thin and large pellicular ice plates are first formed and they drift slowly in the lake. They break when then get to higher velocity areas in turbulent flow, or when they hit the piers of the bridges. Smaller but thicker ice plates also are released from border ice, with mild weather or wind. Finally then, for all weather conditions, there are always ice plates which are feeding the Lachine rapids from the very long limit of shorefast ice in lake St. Louis. This produces, at times, very considerable amounts of drifting surface ice.

The last type of ice which has been observed is anchor ice (Fig. 5) which detachs from the bottom and drifts to the surface. The maximum quantity of this ice was seen during very sunny periods for two days in February. It was seen occasionally, at other times. These floes are made of a mass of thin platelets of ice crossed in different directions, but usually with a prefered orientation. They transport bottom material, one clam shell and one piece of rock, 15 cm diameter, were observed in them. Aerial photographs show where they are formed. The upstream limit is at the outlet of lake St. Louis and it corresponds very closely to areas where water supercooling begins to occur. The thickness of these floes has been estimated to be somewhat over 1 m.

Concentration of drifting ice

The concentration of drifting ice was obtained by oblique pictures all along Mercier bridge. These pictures were taken every hour during daylight time. The pictures were projected on a screen with distorted squares to evaluate the correct concentration of surface ice in each one. The integration of each square concentration, across the river, gave the total concentration.

It was interesting to discover that surface ice concentrations were always high, even during warm and melting temperatures. This is due to the fact that the ice plates which are formed on lake St. Louis, detach in more considerable quantities during warm weather. For these conditions a maximum concentration of 56% for one hour was measured at Mercier bridge and 52% for a whole day. This ice is, however, rather thin and can be more easily evacuated over the projected ice spillway. The thickest ice formed during the winter was the drifting anchor ice, with an average thickness of 1 m. The maximum concentration could attain 20% for 2 hours at Mercier bridge.

The ice breakup in the spring is not expected to cause problems, because most of the ice will be retained in place by the booms and also because the flood flow could be used to by-pass this ice without interruption of power production.

Computation of drifting plate ice

An equation was established to represent the concentration of drifting ice plates at Mercier bridge in function of the rate of heat exchange ϕ in kcal/m^{2}day. There are two conditions of ice production. The first condition occurs when the main channel in lake St. Louis remains opened and plate ice is being produced both in this channel and in a large flowing water expanse in Dorval's Bay. The second condition is when the channel is closed and only Dorval's Bay is producing plate ice.

The concentration C at the bridge is obtained from continuity considerations:

$$C = \frac{\phi_1 \, (t - 8) \, B_1 \, V_1/\phi_{1_{limit}} + \phi_2 \, (t - 5) \, B_2 \, V_2/\phi_{2_{limit}}}{B \, \bar{V}} \qquad (1)$$

where: C, B and $\bar{V}$ are the concentration, width and average velocities at the bridge

B_1 V_1 and B_2 V_2 are the width and average velocities at the outlet of the channel (zone 1) and Dorval's Bay (zone 2)

ϕ_1 and ϕ_2 are the heat exchanges at time (t-8) hours in zone (1) and (t-5) hours in zone (2)

$\phi_{1_{limit}}$ and $\phi_{2_{limit}}$ are the heat exchange giving a concentration of one at the outlet of each zone.

The maximum concentration at the bridge are obtained when the concentrations are equal to one at each zone outlet.

This gives computed maximum concentrations:

C = 0,66 (open channel)
C = 0,38 (close channel)

These values are close to those which were observed at the bridge.

Concentration of drifting frazil slush

The drifting frazil slush concentration is maximum at night and could not be measured.

Its evaluation was made only by computation of frazil production from heat exchanges. The frazil volume in suspension in the flowing water which was estimated from measurements, had to be substracted.

The basic computation is made in the following manner.

Let us consider a length Δx of a uniform flow, which water travels in time Δt, and where the surface discharge of ice is ΔQ_i at the entrance (concentration C) and $Q_i + \Delta Q_i$ at the outlet (concentration $C + \Delta C$), the quantity of ice produced over the length Δx of width B is:

$$\Delta Q_i = \frac{B \Delta x \, (1 - C) \, \phi}{L} \tag{2}$$

where: ϕ - heat exchange in $kcal/m^2 s^{-1}$
L - latent heat of fusion of ice

Part of the frazil formed $(1 - \nu) \, \Delta Q_i$ floats to the surface to form a slush of thickness h', whose non-porous equivalence is h. The increase in surface ice concentration is thus:

$$\Delta C \; h \; B \; \Delta x = (1 - \nu) \, \Delta Q_i \, \Delta t \tag{3}$$

where ν is the fraction of slush in suspension estimated by measurements to be, on the safe side, 0,16 under Mercier bridge.

Eliminating Q_i between (2) and (3):

$$\frac{\Delta C}{1-C} = \frac{(1 - \nu) \, \phi \, \Delta t}{L \, h} \tag{4}$$

The integration gives:

$$C = 1 - e^{-kT} \tag{5}$$

with $k = \frac{(1 - \nu) \, \phi}{L \, h}$

where T is the total drift time. The concentration is zero at time T = 0 where the frazil front starts and attains unity at an infinite distance.

Table 1 shows the concentration of slush at the surface for various values of heat exchange rate ϕ and drift time T. We assumed that the slush is 15 cm thick and has a porosity of 0,33; i.e., h = 5 cm.

It takes a long time to get important surface ice concentrations. For the dam open forebay, with an extreme heat exchange value of $-21\,000$ $kcal/m^2 day$, and a concentration time of 2,4 hours required to drift slush from the projected ice boom, the surface concentration would be only 36,5% at the site of the spillway and the slush pans would have thickened only to 5,7 cm.

T (h) / ϕ Kcal/m²day	1	2	3	4
1 000	0,01	0,02	0,03	0,03
2 000	0,02	0,03	0,05	0,07
5 000	0,04	0,08	0,12	0,16
10 000	0,08	0,16	0,22	0,29
15 000	0,12	0,23	0,32	0,40

Table 1 - Concentration of frazil slush for various travel times.

FORMATION AND ADHESIVITY OF ACTIVE FRAZIL

Methods of measurement

Measurements on the formation and deposits of active frazil were made from an experimental station built under the lower trusses of Mercier bridge (Fig. 6). It is made of a temporary tunnel where the target objects could be lowered into the water. An enclosed station was also built where a cage could be lowered into high velocity water, with a winch and cables. The cage had a cubic form whose sides were 1,5 m long. There was also small wings on each side (30 cm wide) to act as classical frazil grids and also as plungers for the whole cage (Fig. 7).

The velocity where the cage was dropped in very high at 1,7 m s^{-1} and the velocity at the bottom (6,3 m of water) is 1,0 m s^{-1}. Because of the continuously drifting ice, these were very severe conditions. In order to maintain the cage in place a wood spur structure had to be built and set floating on top, attached to the neighboring bridge pier, to deflect the drifting ice in front of the cage (Fig. 8).

Formation of active frazil

Active frazil is formed as soon as there is a very slight supercooling of water. Numerous observations proved that this occured at night, at the Mercier bridge and most probably at the future power plant, in the form of residual supercooling with active frazil formation. Temperature measurements have shown that the front of frazil production (with more supercooling) is formed at the outlet of lake St. Louis and moves upstream with a lowering of external temperatures (Fig. 9). The supercooling measured at the bridge was usually around 0,01°C; sometimes 0,02°C and it attained occasionnally 0,03°C.

Figure 10 shows a correlation for one week between air temperatures at the bridge and the deposits of active frazil on the wing grids of the cage. The cage is brought up every four hours and, except for one night, when the deposit was dislodged, there is a deposit each time that the water temperature is supercooled. The end of the period of formation is characterized by an increase in air temperature which makes the deposit melt.

It is remarkable to notice that active frazil forms even when the temperature is not much under 0°C. The emitted nightly radiations are extremely favorable for cooling the water and forming active frazil.

On the other side, even if the temperature is below 0°C during daytime, the absorption of solar radiations in the morning compensates the heat gain by convection and no active frazil is formed.

Adhesion of active frazil

Many types of targets have been used to study the adhesion of active frazil. First, targets made of concrete plates were made to simulate the intake walls to the turbines. Then, a steel propeller, 50 cm in diameter, was used at different speeds to simulate a turbine blade. Finally, targets of different forms and materials were used on a comparative basis (Figs 11 and

During the month of February, active frazil accumulations were observed during 17 nights. The cage was put down at sunset and taken up every 4 hours until dawn. The maximum deposits were observed during two consecutive nights. The first one, a 7 cm deposit, was measured on the reference grids of the cage when the minimum air temperature was -9°C at night. The maximum deposit was 9 cm on the grid, the following night, with a minimum air temperature of -7°C.

Various types of concrete plates were tested during that monti: ordinary, high resistance and epoxy types concrete, but no differe:ce was seen between one or the other. The deposits were never thicker than 6 cm.

The tests on the propeller were made at two different speeds. At 430 rpm a compact deposit was observed to a distance of 7 cm from the center hub. By reducing the speed to 215 rpm a small deposit of frozen frazil was observed to a distance of 20 cm from the hub.

Deposits on other targets were measured regularly. The cage itself was a target; its frame bars and supporting chains. These chains were the best frazil detectors. Deposits having a total diameter between 5 and 25 cm were measured, made, in general of a relatively soft frazil made of fine grain accumulations.

Individual targets were set, every day, in zones of low flow velocity close to the bridge piers. The maximum measured deposits at low water velocities were the following:

- ice cylinder - 3 cm
- small grate 30 cm x 30 cm - 10 cm
- steel plate 15 cm x 15 cm - 10 cm
- steel cable 8 mm - 18 cm ϕ
- concrete cylinder 20 cm ϕ - 10 cm
- nylon cable - 20 cm ϕ.

Three types of frazil deposits were observed. The first one was like a hard rime, compact with no porosity. The second one was a compacted five grain slush, relatively soft and water retaining. The third was an open structured mass of crossed platelets, each up to 1 cm in length.

Effect of velocity on frazil deposits

Although it was thought that increasing velocities would increase frazil deposits, the contrary was observed, at least at higher velocities.

Thus the following measurements were made during one night at increasing velocities:

- 1 m s^{-1} - maximum deposits of 9 cm on the cage gratings, 10 cm on the frame bars and 12 cm on steel chains
- 1,5 m s^{-1} - maximum deposits of 10 cm on a nylon cable, a cylinder, a small grid and a small steel plate
- 1,7 m s^{-1} - on the same individual targets, maximum deposits of around 8 cm
- 2,0 m s^{-1} - maximum deposits of 5 cm on individual targets
- 4,6 m s^{-1} - no adhesion of active frazil on the steel propeller.

Growth of anchor ice

Anchor ice was found to be the most undesirable form of ice that may affect the project. Some hypothesis on its possible growth were proposed verified by a few measurements.

All observations have shown that there are three mecanisms of growth of anchor ice. It can grow in the form of platelets extending by thermal growth into the supercooled water, it can be compacted with no or little contribution from drifting frazil and finally it can be formed as an underwater rime.

Growth of ice platelets

Crystal growth of ice needles has been studied by a few authors and is documented up to velocities of 50 cm s^{-1} (Vlahahis and Barduhn (1974)). It can be expressed by:

$$\frac{\Delta h}{\Delta t'} = 0{,}0466\ U_l^{0,5}\ \Delta\theta^{1,5} \qquad (6)$$

where $\Delta h/\Delta t'$ is the growth rate in cm s^{-1}, U_l the velocity in cm s^{-1} and $\Delta\theta$ the amount of supercooling in ^{o}C. With the value $\Delta\theta = 0{,}01^{o}C$ which we have most continuously observed during supercooling, we have:

$$\frac{\Delta h}{\Delta t} = 1{,}68\ U^{0,5} \qquad (7)$$

where Δt is in hours and U in m s^{-1}.

As there is no possible supercooling in laminar flow, i.e. for velocities $U \leq 0{,}2$ m s^{-1} this relation must be verified for:

$0{,}2$ m $s^{-1} < U < 0{,}5$ m s^{-1}

Underwater rime

The growth of rime at the river bottom can be obtained directly from the general heat exchange formula:

$$Q' = \frac{\Delta h'}{\Delta t'} \rho' L = H\Delta\theta \qquad (8)$$

where: ϕ - rate of heat exchange KJ/m^2 s
$\Delta h'/\Delta t'$ - ice growth in m s^{-1}
ρ' - specific mass of ice (900 kg/m^3)
L - latent heat of fusion of ice (3,34 x 10^2 KJ/kg)
H - coefficient of heat exchange KJ/m^3 s ^{o}C
$\Delta\theta$ - temperature of supercooling in ^{o}C.

The heat exchange coefficient can be taken as (Cowley and Lavender (1975)):

$$Nu = 23,4 \frac{n^2}{Y^{1/3}} Re \qquad (9)$$

where the Nusselt number Nu is:

$$Nu = \frac{HY}{k} \qquad (10)$$

where: k - water conductivity at 0^{o}C (0,55 J/m s ^{o}C)
Y - water depth in m
Re - Reynolds' number

$$Re = \rho \frac{U\,Y}{\mu} \qquad (11)$$

where: μ - dynamic viscosity of water at 0^{o}C (1,79 x 10^{-3} kg/m s)

With the numerial values of the parameters, a Manning's number, n = 0,02, an average water depth of 5 m and a supercooling of 0,01^{o}C we obtain:

$$\frac{\Delta h}{\Delta t} = 2 \times 10^{2}\, U \qquad (12)$$

where: $\Delta h/\Delta t$ - is in cm hr^{-1}
U - is in m s^{-1}.

This type of ice has been observed to form at velocities over 3,3 m s^{-1} on the cage propeller.

Erosion and compaction

Between an estimated velocity of 0,7 m s^{-1} where the growth of ice platelets is dominant and 3,3 m s^{-1} where only rime is observed, the growth can only occur by compaction of the ice formed in place to resist erosion. Table 2 summarizes typical experimental results obtained during the study.

U m s^{-1}	$\Delta h/\Delta t$ cm/h	Remarks
1,0	1,27	wings of the cage
1,5	0,71	individual targets
1,7	0,51	individual targets
2,0	0,35	individual targets
4,6	0	propeller

Table 2 - Rate of growth of anchor ice in the compaction zone.

A diagram is shown on Figure 13 giving the growth rate of anchor ice in function of flow velocities. For velocities from 0 to 0,2 m s^{-1} there is no anchor ice. For velocities between 0,2 to 0,7 m s^{-1} the rates are increasing from 0,73 cm h^{-1} to 1,41 cm h^{-1}. From 0,7 m s^{-1} to 3,3 m s^{-1} rates are decreasing gradually to 0,07 cm h^{-1}, and finally at 4,6 m s^{-1} there is no ice.

Application to the Lachine rapids

It is interesting to compute the thickness of anchor ice at the outlet of lake St. Louis where the velocity is around 1,5 m s^{-1}.

The longest period of undercooling occurred for 138 h from January 14 to January 20, 1983. This gives a computed growth of 1,38 m of anchor ice. This value is consistant with the thickness of the anchor ice that floated down this area on two sunny days in February.

OTHER MEASUREMENTS

Many other measurements were made which cannot be discussed here:
- all the elements of the thermal budget
- velocity profiles
- frazil concentration profiles
- velocity of equilibrium of shorefast ice
- formation and equilibrium of ice cover in lake St. Louis channel
- temperature distribution.

These measurements are discussed in the final report (Michel et al., 1983).

CONCLUSIONS

The winter 1982-83 was exceptionnally short and warm, in fact the warmest on record in the Montreal region so that the results may not represent the worst cases for the operation of a power plant.

Insofar as the surface ice discharge is concerned, measurements and their interpretation give an excellent method to predict the maximum ice discharge to be expected. It would be lower than under natural conditions because of a retaining boom to be set in lake St. Louis. It would be thin ice that could be passed over the spillway with a small water depth and a small water discharge. This is seen as rather easily feasible.

As for active frazil deposits it seems that the thicknesses of the deposits depend on four major factors: rate of supercooling, duration of supercooling, velocity of flow and finally form and physical characteristics of the targets.

An analysis was made of the first three factors to obtain the rate of growth of anchor ice which seems to be maximum at velocities of around $0,7\ m\ s^{-1}$. If there is sustained undercooling in the opened flow for a long time, thick anchor ice might be formed. Unfortunately all measurements were made, one night at a time only, so the total growth of anchor ice was never measured.

The form of the object, on which active frazil deposits, has also a large importance. Deposits are thin on smooth surfaces parallel to the flow and increase considerably on rounded objects placed perpendicular to the flow. Finally the deposits never exceeded 6 cm on various concrete plates but were much thicker on steel bars and chains.

Although much was learned from this field survey it was concluded that new methods should be developped to study "in situ" the growth of anchor ice over a period of supercooling of many days.

REFERENCES

COWLEY, J.E. and LAVENDER, S.T., (1975), "Convective heat transfer at an ice-water interface" Proc. Research Seminar Thermal Regime of River Ice, Tech. Memo. No. 114, N.R.C., p. 60-78.

MICHEL, B.C., (1983), "Archipel: Résultats de la campagne de mesures des glaces, hiver 1982-83" Rapport à Hydro-Québec, 226 p. et 3 annexes.

VLAHAKIS, J.G. and BARDUHN, A.J., (1974), "Growth rate of an ice crystal in flowing water and salt solutions" Am. In. Chemical Eng. Journal, Vol. 20, No. 3, pp. 581-590.

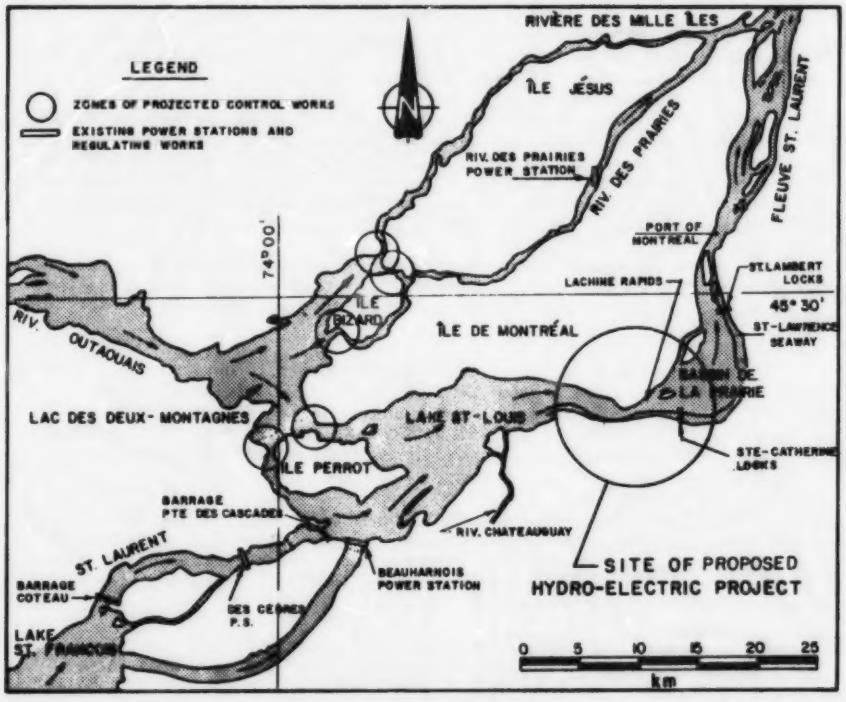


FIGURE I

THE MONTREAL ARCHIPELAGO

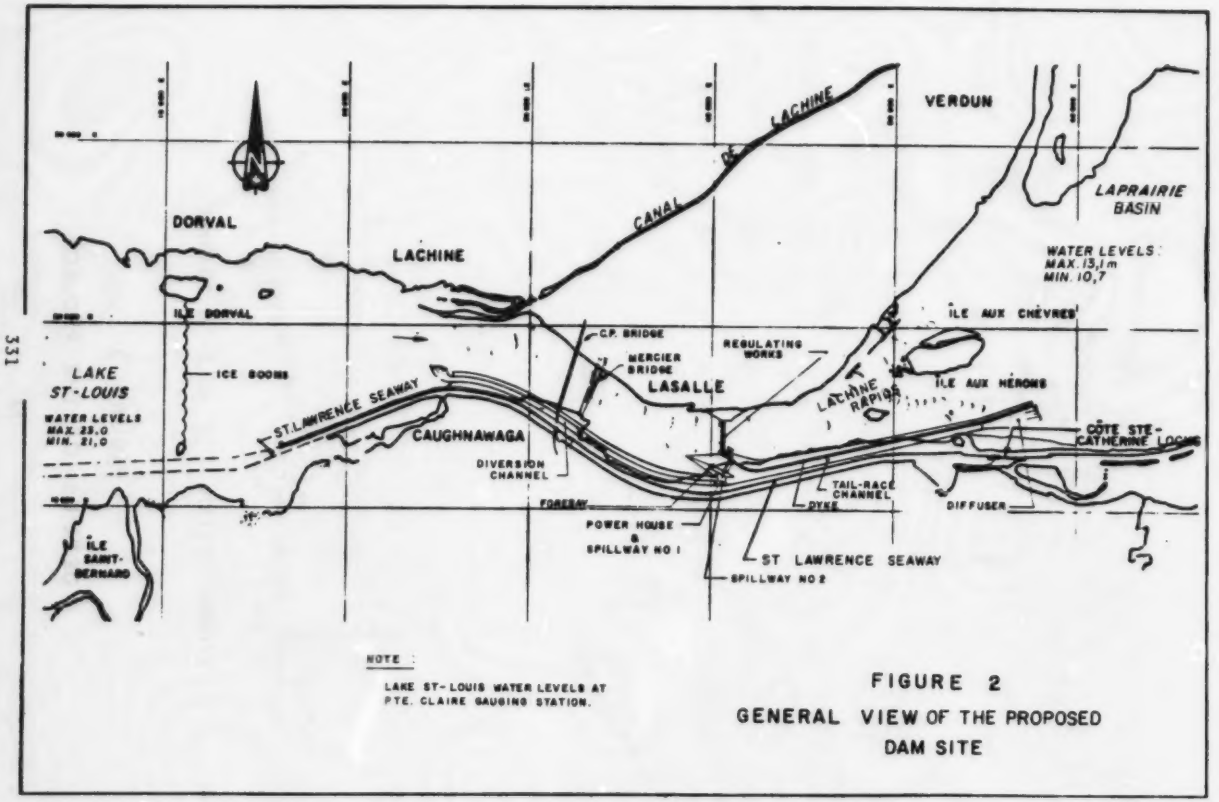
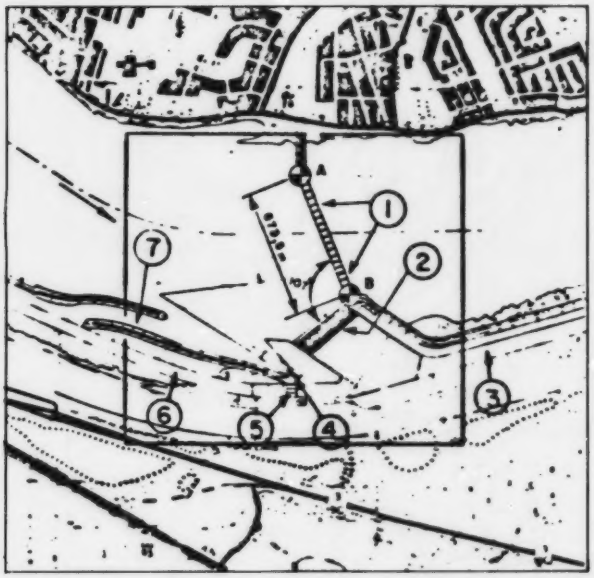


FIGURE 2
GENERAL VIEW OF THE PROPOSED DAM SITE



1 REGULATING WORKS

2 AQUEDUC POWER STATION
6/12/16 BULB TURBINES

3 TAIL RACE CHANNEL

4 SPILLWAY NO 1

5 SPILLWAY NO 2

6 DIVERSION CHANNEL

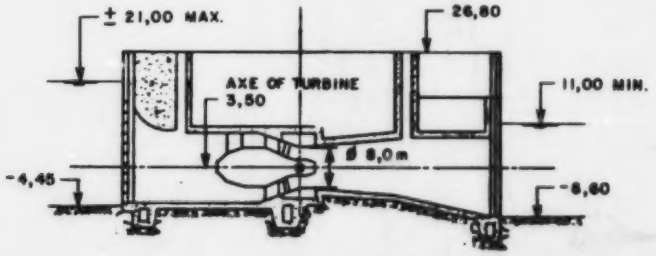
7 INTERCONNECTING WORKS

COORDINATES

A 14 995,0 N
39 727,0 E

B 12 933,0 N
40 520,0 E

1 : 25 000



TYPICAL SECTION OF BULB TURBINE

FIGURE 3

DETAILS OF THE PROJECT

A - Large ice plates being formed in the lake and drifting down.

B - Ice plates breaking down in small pieces and appearance of frazil in the rapids.

FIGURE 4
Ice formation at the head of the Lachine Rapids

A - Large quantity of frazil slush appearing downstream from the rapids in the south channel of St. Helen island.

B - Anchor ice floes moving up on sunny days at the head of the rapids.

FIGURE 5
Types of ice

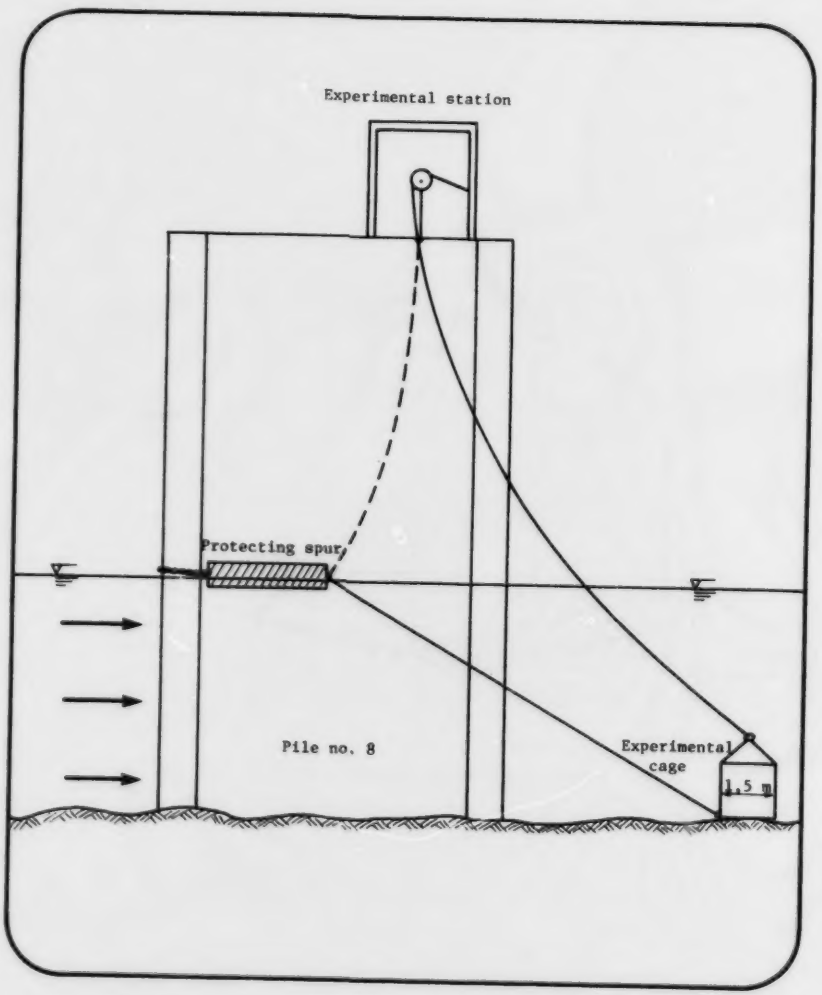


Figure 6 - Experimental station under Mercier bridge

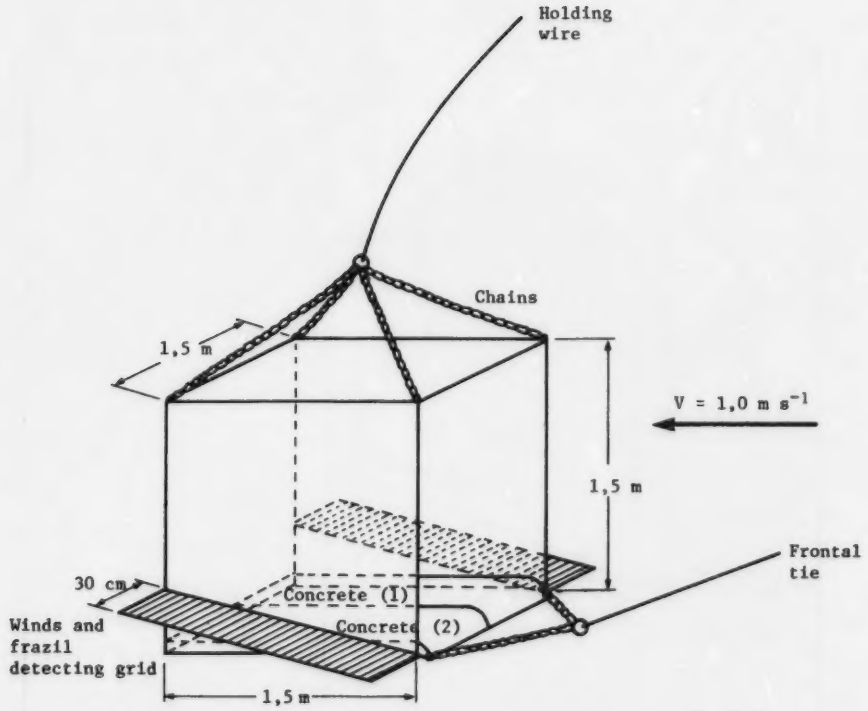


Figure 7 - The experimental cage

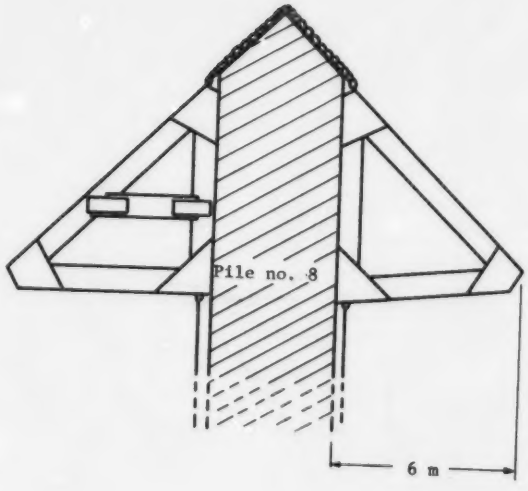


Figure 8 - Wooden spur protector

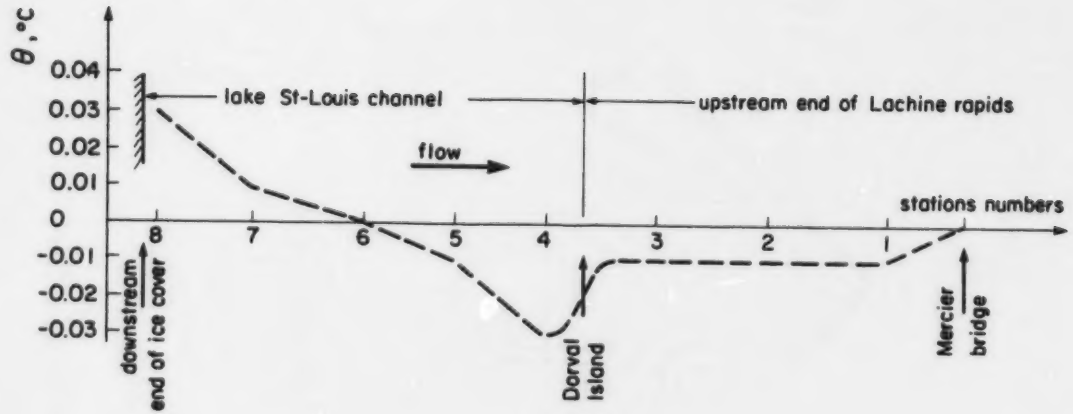


FIGURE 9

Evolution of water temperature along the axis of the St. Lawrence river from the channel in lake St. Louis to the Lachine rapids at time of frazil formation. The distance between each station is about 1 km.

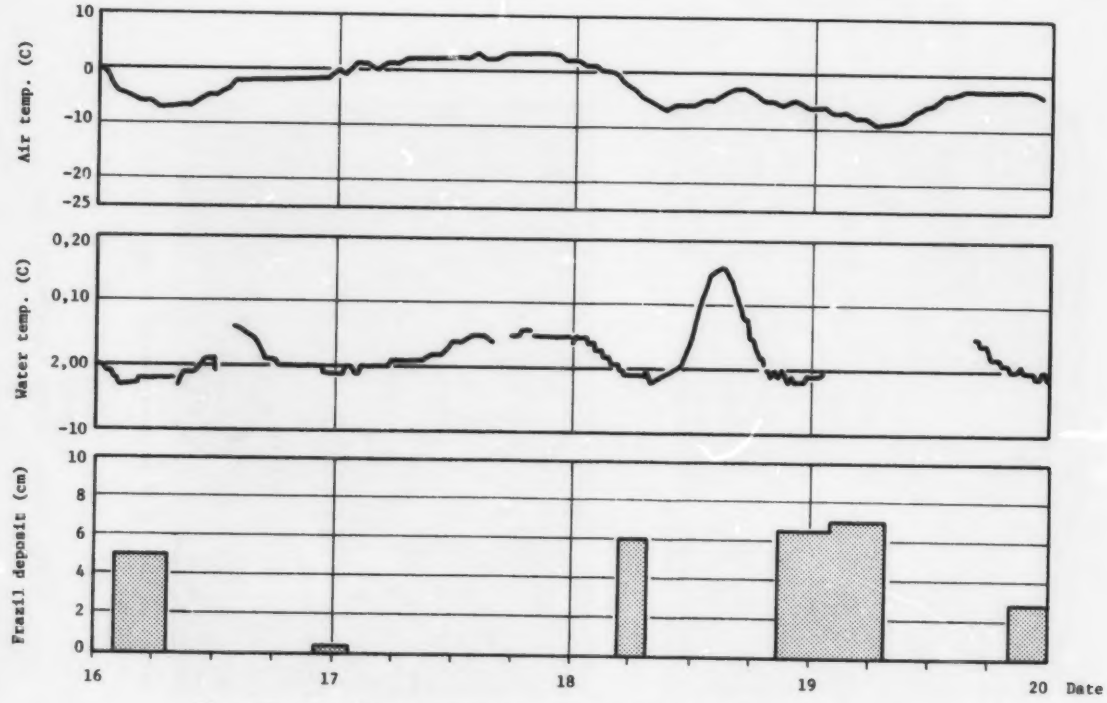
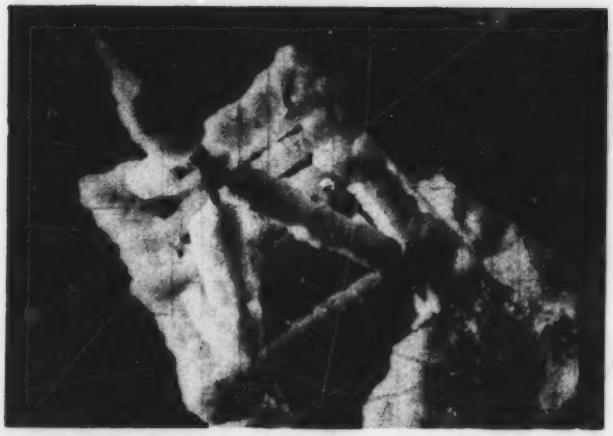


Figure 10 - Correlation between air temperatures, water temperatures and frazil deposits from Feb. 16 to Feb. 20, at Mercier bridge

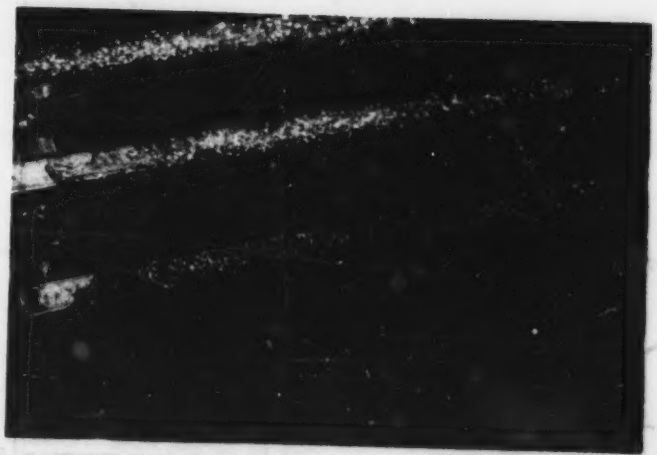


A - Deposit of soft frazil on the frame of the cage (no deposit on bottom concrete plates).

B - Frazil on the hub of an experimental propeller rotating at 430 rpm.

FIGURE 11
Frazil deposits on the experimental cage

A - Deposit on a rope and a small reference grid.



B - Underwater rime formation on a cage wing grid.

FIGURE 12
Types of frazil deposits

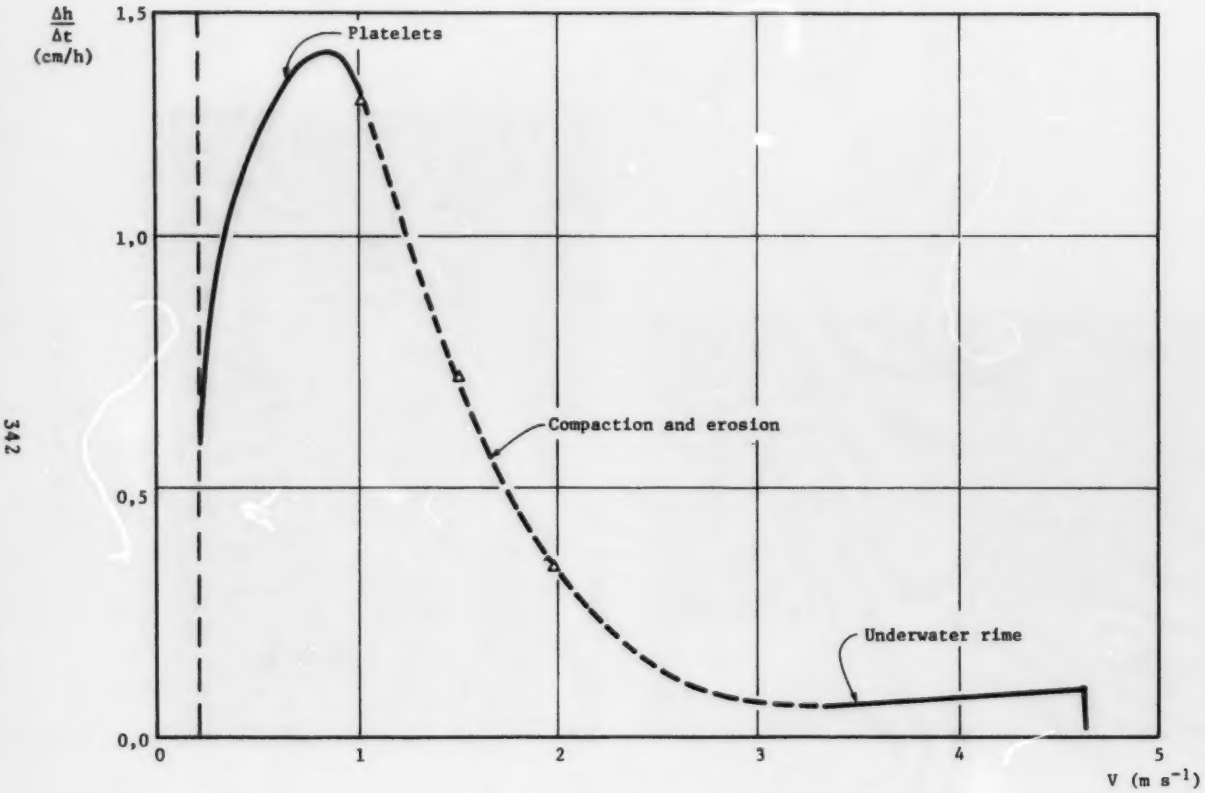


Figure 13 - Growth of anchor ice

DISCUSSION

Unnamed

As regards to more frazil being deposited on the steel chains compared to that on concrete, could this be attributed to the difference in heat capacity of steel and concrete?

Reply

We believe it would depend much more on surface properties and form, than on heat capacity. Some surface properties are related to heat capacity.

D. Foulds

In support of the validity of reducing the flow velocities, we can state that the Sir Adam Beck #1 G. S. at Niagara has worked without frazil problems for 50 years with velocities of 2.5 to 3 m/s.

Reply

This fact was often taken as an example for this project. However, there is a difference in that the long free surface channels leading to the turbines at Sir Adam Beck #1 G. S. almost always have water temperatures above the freezing point, so there is no possible frazil effect on the machines.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

ANCHOR ICE EFFECTS ON WATER LEVELS IN LAKE ST.LOUIS, ST.LAWRENCE RIVER AT MONTREAL

F.E. Parkinson[1]

ABSTRACT

A limited two-year survey program on Lake St.Louis, on the St.Lawrence River at Montreal, has included measurements of water levels and temperatures as well as observations of anchor ice formation, growth and release. Meteorological recordings of air temperature, wind and hours of sunshine were obtained from the Atmospheric Environment Service at nearby Dorval Airport. Finally, the discharge through the lake was provided by Hydro-Quebec from their mathematical model of this reach, including known discharges from controlling upstream hydroelectric plants.

Interpretation of these data was directed toward eliminating water level changes due to discharge variations and the surface ice cover, leaving only the effects of the anchor ice. The anchor ice effects were evaluated as a function of the field observations and atmospheric conditions.

Final results suggest that the water level in the lake reacts very quickly to the formation or release of anchor ice.

[1] *Lasalle Hydraulic Laboratory, Lasalle, Quebec*

RESUME

Une campagne de mesure limitée à deux ans d'observations sur le lac Saint-Louis (fleuve Saint-Laurent à Montréal) a porté sur les niveaux d'eau, les températures de l'eau ainsi que sur la formation, l'évolution et le décollement des glaces de fond. Les données météorologiques comprenant la température de l'air, la vitesse des vents et les durées d'ensoleillement ont été obtenues du service de l'Environnement atmosphérique de l'aéroport de Dorval. Quant aux débits, ils ont été fournis par Hydro-Québec qui les a déterminés à partir de l'exploitation des usines hydroélectriques situées en amont du lac.

L'interprétation des résultats a consisté à séparer l'influence des changements du débit sur les niveaux d'eau de façon à caractériser les effets de la glace de fond sur les niveaux; ces effets étant rattachés aux conditions atmosphériques.

Les résultats indiquent que le niveau du lac réagit très vite à la formation ou au décollement de la glace de fond.

INTRODUCTION

The possibility of developing the hydroelectric potential available at the Lachine Rapids has been studied a number of times over the past 100 years, but the most recent investigation was the first to combine such a project with flood control measures around the Island of Montreal. The scope of these studies was expanded to include control works between Lake St.Louis and Lake of the Two Mountains, as well as at the entries to the Mille Iles and des Prairies Rivers, which exit from Lake of the Two Mountains.

With these expanded interests, water level control was more important, and understanding the influences of all factors acting on the levels became necessary. It has been known for some time that anchor ice occurs regularly in Lake St.Louis and down into the head of the Lachine Rapids, but no quantitative values were available concerning the meteorological conditions under which it is generated or released, or to what extent the lake level is actually modified.

The present study in the form of a private research project undertaken by the Lasalle Hydraulic Laboratory during the winter of 1983-84 has made a modest effort at identifying some of these parameters.

STUDY AREA

Most of the St.Lawrence River flow enters Lake St.Louis through the Beauharnois Canal and powerplant, with smaller amounts passing through the original riverbed past Coteau-du-Lac as shown on Figure I. The Ottawa River enters the Lake of Two Mountains, from which its waters are distributed partly into Lake St.Louis on either side of Ile Perrot, and partly into the Mille Iles and des Prairies Rivers, to the north of Montreal Island.

Water originating in the Ottawa River sticks to the left shore on its way through Lake St.Louis, and reaches out approximately one quarter of the lake width. The remaining three-quarters is filled with water from the St. Lawrence River. The two streams remain fairly distinct all the way through the lake, being visibly of different colours, and having different water qualities and temperatures.

Three observation sites were selected for this study, all of them along the left shore of the lake, at Dorval Avenue (DA), 51 st. Avenue (51) and at the Transport Canada Dock (TD) at 32nd

Avenue in Lachine. Their locations are shown on Figure II. Water Survey of Canada (WSC) level recording gauges are installed at Pointe-Claire, Lock No. 5, and Old Aqueduct, and Hydro-Quebec had a number of gauges through the reach. Meteorological data was obtained from the Atmospheric Environment Service at Dorval Airport, also shown on Figure II. The stage-discharge curve (see Fig. III) corresponding to levels at the WSC gauge at Lock No 5 was used for open water stage-discharge interpretations.

Figure II shows the approximate downstream limit of the ice cover on Lake St.Louis, during the 1983-84 winter, indicating that the three observation sites were on open water all winter. Although the term lake is used for this body of water, it must be kept in mind that the whole St.Lawrence River plus some of the Ottawa River is flowing through it. Water velocities in the open water zones along the left shore where the observations were made varied depending on discharge and location from 0.3 to 1.0 m/s.

FIELD DATA COLLECTED

Daily observations were made at each of the three sites along the left shore of Lake St.Louis. Normally, the first visit was between 7:30 and 8:30 in the morning, when the water levels, and air and water temperatures were recorded. Notes were also made of the presence of anchor ice, either on the lake bed, on rocks along the shore, on the shore ice, or floating by in the open water. On days when there was anchor ice activity of particular interest, additional visits to the sites were made during the day and early evening. Telephone calls were made daily as well to the Telemark recorders at the WSC gauges at Pointe-Claire and Old Aqueduct.

During the interpretation studies, the monthly climatological resumés at Dorval were obtained from AES. Hydro-Quebec furnished recordings of water levels at Pointe-Claire as well their calculated values for the discharge going through the Lachine Rapids, and finally the St.Lawrence River water temperatures as measured at the Valleyfield bridge.

WATER TEMPERATURES

Thermometers used during these observations, both mercury and electronic, were capable of reading temperatures only to 0.1°C accuracy. Therefore, no data was gathered concerning supercooling of the water.

However, the temperatures that could be taken did at least identify the periods when the water was at 0°C, so susceptible to being supercooled to generate frazil if the appropriate meteorological conditions occurred. Figure IV shows the values recorded for the two winters, 1982-83 and 1983-84.

During the 1982-83 campaign, a very sharp difference in temperatures between the Ottawa and St.Lawrence River waters appeared. The St.Lawrence dropped to 0°C for only 44 days, while the Ottawa stayed down for 84 days. However, Figure IV points out that this past winter, the Ottawa was at 0°C 116 days, just 13 days more than the St.Lawrence.

No firm conclusion can be drawn from this data for the moment, beyond the general statement that anchor ice would exist longer in the Ottawa water than in the St.Lawrence water. This in turn could lead to increased diversion of Ottawa water toward the St.Lawrence, resulting in more mixing, during these periods of early and late extra anchor ice formation.

On the positive side, knowing that there may be anchor ice on one side of the river and not the other may ultimately aid in interpreting water levels across, or through the reach. At the present time, sufficiently accurate water level measurements like this do not exist, but future work should take these parameters into account.

LAKE LEVEL FLUCTUATIONS

Daily water level readings at the three sites, when combined with observations of the anchor ice conditions, were adequate to identify the general effects of anchor ice, and the likelihood of its modifying the lake level. However, a more precise continuous recording of the water levels was necessary to interpret the effects quantitatively. Figure V shows the water levels at Pointe-Claire as recorded by the Hydrological Service of Hydro-Quebec that were used.

In order to be able to identify the anchor ice effects, it was necessary to know what the discharges through the lake were so their influences could be taken into account. The same group in Hydro-Quebec provided the discharge at Lachine Rapids as shown on Figure VI, which is the result of a mathematical model that includes the known discharges from the Beauharnois and Carillon hydroelectric plants, plus gauged inflows from other smaller streams between the plants and the rapids.

Using the levels presented on Figure V along with field notes describing anchor ice occurrences, dates were selected when either

rapid rises were noted suggesting anchor ice formation, or rapid drops suggesting anchor ice release. Reference to Figure VI on the selected dates provided the corresponding discharges, for which the controlling open water levels were taken off Figure III. The use of the open water curve is justified since, as shown on Figure II, virtually all of the reach under study was free of a fixed ice cover. Taking the sums or differences of those two values results in the effects introduced by the anchor ice, whose cumulative values are plotted on Figure VII along with field observations of anchor ice occurrences.

Table I shows the interpretation procedure where the actual rises and drops over selected intervals measured on the lake are compared with the level changes that would have occurred due to the discharge changes in open water.

Although the field observations do not always coincide precisely with the expected rise or drop in lake level, most of them do show a rise during formation and a drop during release.

Another indicator of the presence of anchor ice in the control section at the outlet of the lake can be developed by comparing the daily water levels at Pointe-Claire and Old Aqueduct gauging stations. Reference to Figure II shows that while Pointe-Claire is well up in the lake, the Old Aqueduct gauge is well downstream in the narrow river section, where the velocity has risen considerably and there has already been a significant drop in level. Figure VIII shows that the mean level difference between the two stations during open water is about 2.33 m. Conversely, during the winter, the level differences varied over a wide range, and comparisons with Figures V and VII show that they corresponded to the periods of anchor ice formation and release identified from the lake level interpretation. Increased level differences between the two stations coincided with rises in the lake level, supporting the argument in favour of the presence of anchor ice in that reach.

Comparisons between the periods of rapid rise or drop due to anchor ice on Figure VII with the air temperatures on Figure IX and hours of sunshine on Figure X also supports the general processes associated with the phenomena. At all times when there were rises due to formation of anchor ice, the minimum daily temperatures were well below freezing and there was generally sunny conditions.

All but one of the major water level drops occurred after thawing air temperatures had been experienced; i.e., January 4-7, January 24-26, February 3-7, February 12-14, March 4-6, March 14-17, and March 19-20. The one exception is January 14-16, when a 0.35 m drop appears to have occurred solely under the influence of solar radiation, as the air temperatures remained below -10°C at all times.

Seemingly contradicting this conclusion, that thawing air temperatures cause the anchor ice to release, were numerous field observations when anchor ice was noted rising on extremely cold days presumably under the influence of the bright sunshine. However, as will be discussed in the following section, these cases were normally associated with net water level rises during a cold period that was gradually building up a significant mattress of anchor ice.

Theoretically the cumulative changes in lake level that were attributed to anchor ice should cancel out at zero once the last anchor ice had left. Figure VII shows that in fact their remained an error of 0.11 m. This does not appear to be too great considering the simplicity of the methodology and the fact that this is the first time that it has been applied. It is possible that a more careful analysis, taking more time than the present one, might reduce the error. Also, succeeding years' surveys and interpretation could be done with more care, and should yield a more accurate closure.

DAILY FLUCTUATIONS

The daily water level fluctuations shown on Figure XI are an excellent example of the result of the formation of anchor ice at night followed by partial release the next morning. Sufficient familiarity with the processes was built up during the survey period so that it was possible to predict these phenomena fairly accurately.

Figure XII shows that the discharge was dropping generally over March 1 and 2 and most of the 3rd, while the level continued its net rise. Figure XIII indicates that air temperatures were well below freezing, and that the skies remained clear most of the time.

Visits to the sites shortly after sunrise each morning revealed new formations of anchor ice on the rocks and shore ice that were accessible along the shore. The water levels had risen. As the morning progressed, the solar radiation penetrated the water sufficiently to release some of anchor ice, so that by 10:30, accumulations of the anchor ice were being released from the bottom, bobbing up to the surface, and floating away downstream. This release would normally continue until about 16:00 hrs in the afternoon, when the levels were noted to have dropped slightly.

The air temperatures remained distinctly below freezing, and the water temperatures measured were 0°C. Therefore, it was reasoned that with the bright sunshine, as on March 1, 3 and 4 (see Figure XIII), the solar radiation was able to penetrate the clear water to heat up the dark lake bed enough to release some of the anchor ice. It would appear that not all of it came away, as the water level continued to rise, suggesting that there remained a cumulative net increase in the obstruction of anchor ice on the bottom.

The reverse process occurred on March 5 and 6, where air temperatures above 0°C caused a net drop in level each day. During the day, the level dropped rapidly, then at night, as the air temperature went below freezing, anchor ice formed, slowing down the rate of descent of the water level. Still, the net effect was negative, bringing the lake level gradually lower.

ANCHOR ICE CHARACTERISTICS

Successive inspections of anchor ice samples revealed a variety of shapes, structures, and dimensions of individual platelets in the formations. At times when the clear night air temperatures dropped below -20°C, platelet thicknesses regularly reached 3 mm, individual crystal lengths reached as much as 250 mm, while the widths never seemed to exceed 70 mm. A typical sample is shown on Photo I. Accompanying these normal cold condition samples, there were always a few pieces made up of smaller platelets; seldom less than 1 mm thick, but only 25 to 30 mm long and 10 to 15 mm wide.

When the air temperatures were only about -5°C to -10°C, and generally following thawing conditions, much finer structured anchor ice was present. In many cases, the individual platelets were tiny, measuring about 0.1 mm thick by 3 mm wide and 10 mm long, as appear on Photo II.

Twelve samples of various formation types were selected and carefully measured to determine the total encumbrance volume of the whole mass. Next, they were melted down and the volumes of water measured. Accepting a 0.92 S.G. for ice, these water volumes were converted to ice volumes and compared with the mass volumes to determine the porosities of the samples. The lowest porosity found was 70.7% and the highest was 83.5%, with the average of the 12 samples being 75.9%.

The open water reach in front of the observation sites is generally fairly shallow, varying from 1 to 3 m in depth. The bottom is dark in colour, varying from black organic deposits to dark grey and green algae on the rocks. Much of the bottom is covered with broken rock, but there are large areas of sea weed as well. Anchor ice samples carrying grass, clams up to 12 cm in length, snails up to 15 mm diameter and gravel stones up to 8 mm diameter were captured as shown on Photos III, IV and V.

Release of anchor ice from the bottom appears to be related to heat input to the lake bed by solar radiation as well as contributions from rain or atmospheric heat. Repeated cases were noted where there were widespread releases of anchor ice when the atmospheric temperatures remained resolutely below -20°C, but bright early morning sunshine shone through the clear, shallow water onto the anchor ice and the lake bed. Photo VI shows an example where anchor ice up to 80 cm thick was rising off the bottom, breaking through a mobile surface skim ice cover up to 10 mm thick that was still actively generating ice under an atmospheric temperature of -16°C.

Conversely, following our traditional February thaw, the water temperature had risen to +0.2°C. Then, the night of February 16, the atmospheric temperature dropped to just -2°C and the next morning, there were good formations of anchor ice at the water surface on the edges of the shore ice and on rocks along the shore. The water at the surface was 0°C and at 80 cm depth, +0.2°C. The current velocity at that point was about 40 cm/sec.

CONCLUSIONS

- A water level rise of up to 0.5 to 0.6 m can occur over a period of 6 or 7 days due to anchor ice formation.

- A water level drop of 0.6 to 0.7 m can occur in 5 or 6 days due to anchor ice release.

- Daily rises and drops of up to 0.3 m can be caused by either anchor ice formation or release.

- The cumulative effect of the anchor ice can give a lake level up to 0.90 m above its open water level for the same discharge.

- The largest water level drops resulting from anchor ice release occurred as a result of thawing periods accompanied either by rain or bright sunshine which presumably raised the water temperature enough to allow the anchor ice to break free.

- One other water level drop occurred with atmospheric temperatures well below freezing, but with bright sunshine. It is suspected that direct solar penetration through the water to the dark coloured lake bottom provided sufficient heat at the ice/bottom anchor point to allow release of the ice.

- During periods with clear skies and cold weather, anchor ice formation at night is rapid and some of it is released under the sun's radiation next morning. However, on successive days, the anchor ice must have a net positive accumulation as the lake level continued to rise.

- Platelet sizes and thicknesses in the anchor ice mass appear to be influenced by the air temperatures, (and likely by the degree of supercooling) cold conditions gave bigger platelets.

- Average porosity of a series of 12 samples tested was 75.9%.

TABLE I - ANCHOR ICE EFFECTS ON LAKE LEVELS

Date	Q Discharge (m^3/s)	Level Diff. from Q (m)	Diff. Level Measured (m)	Anchor Ice Effect (m)	Cumulative Anchor Ice Effect - m
83/12/05	9,000				
83/12/16	10,000	+0.30	↑ 0.62	+0.32	0.91
83/12/25	7,400	+0.83	↓ 0.82	-0.01	0.90
84/01/02	8,000	+0.19	↑ 0.78	+0.59	0.59
84/01/07	7,600	-0.15	↓ 0.83	-0.68	0.22
84/01/14	7,300	-0.06	↑ 0.45	+0.51	0.73
84/01/20	7,900	+0.19	↓ 0.16	-0.35	0.38
84/01/24	7,900	0	↑ 0.28	+0.28	0.66
84/01/26	7,900	0	↓ 0.37	-0.37	0.29
84/02/03	8,000	+0.02	↑ 0.47	+0.45	0.74
84/02/07	8,300	+0.11	↓ 0.45	-0.56	0.18
84/02/12	8,200	-0.05	↑ 0.45	+0.50	0.68
84/02/14	8,400	+0.09	↓ 0.40	-0.49	0.19
84/02/17	9,000	+0.19	↑ 0.33	+0.14	0.33
84/02/18	8,900	-0.02	↓ 0.11	-0.09	0.24
84/02/20	8,900	0	↑ 0.03	+0.03	0.27
84/02/22	9,600	+0.20	↑ 0.20	0	
84/02/24	9,500	-0.03	↓ 0.20	-0.17	0.10
84/02/27	9,300	-0.05	↑ 0.24	+0.29	0.39
		0	↓ 0.21	-0.21	0.18

TABLE I - ANCHOR ICE EFFECTS ON LAKE LEVELS

Date	Q Discharge (m^3/s)	Level Diff. from Q (m)	Diff. Level Measured (m)	Anchor Ice Effect (m)	Cumulative Anchor Ice Effect - m
84/02/29	9,300				
		+0.12	+ 0.32	+0.20	0.38
84/03/04	9,750				
		-0.19	+ 0.30	+0.11	0.29
84/03/06	9,200				
84/03/12	8,800				
		0	+ 0.23	+0.23	0.50
84/03/14	8,800				
		+0.05	+ 0.22	-0.27	0.23
84/03/17	8,900				
		-0.03	+ 0.10	+0.13	0.36
84/03/19	8,850				
		+0.04	+ 0.21	+0.25	0.11
84/03/20	8,950				

ANCHOR ICE IN LAKE ST.LOUIS
ST.LAWRENCE RIVER, MONTREAL

Photo I: Large Platelet Formation Typical of Cold Weather. 84/02/08 - Ta = -20°C.

Photo II: Small Platelet Formation Typical of Warm Weather. 84/02/08 - Ta = -20°C.

ANCHOR ICE IN LAKE ST.LOUIS
ST.LAWRENCE RIVER, MONTREAL

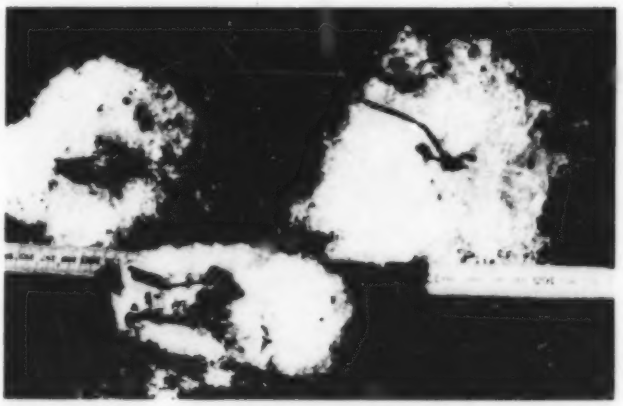


Photo III: Inclusions of Twigs, Snails and Grass confirms origin of ice on lake bottom.
84/03/08 - Ta = -16°C.

Photo IV: Clam Shell Lifted from Lake Bottom by Anchor Ice - 84/02/08 - Ta = -20°C.

ANCHOR ICE IN LAKE ST.LOUIS
ST.LAWRENCE RIVER, MONTREAL

Photo V: Twigs and Grass from Lake Bottom Carried Away by Anchor Ice - 84/03/08 - Ta = -16°C.

Photo VI: Anchor Ice Accumulations up to 0.8 m Thick Releasing from Bottom, Breaking Through Skim Ice up to 10 mm Thick - 84/03/08 - Ta = -16°C, Sunshine.

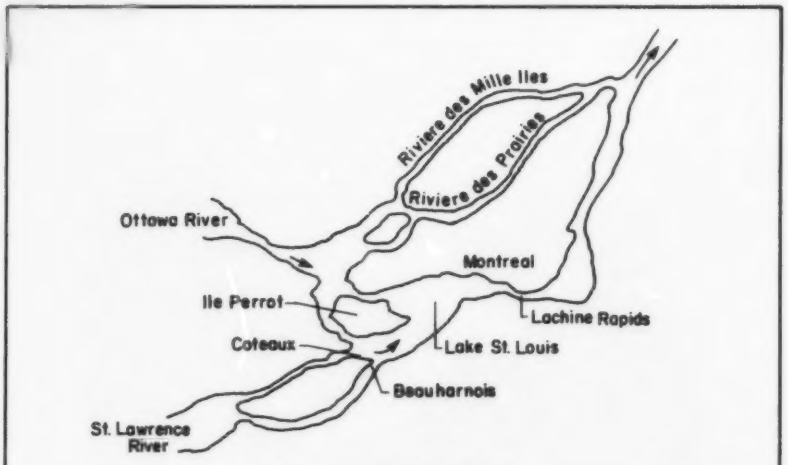


Figure. I Regional Map

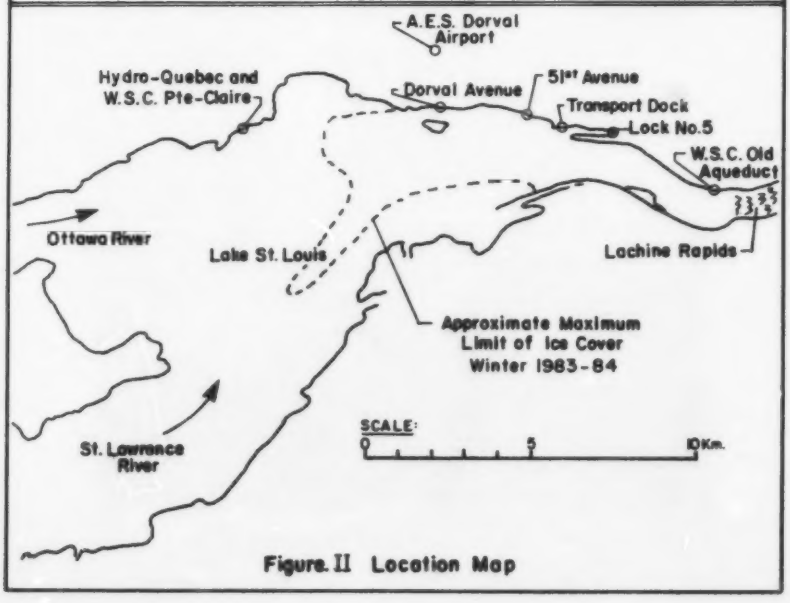


Figure. II Location Map

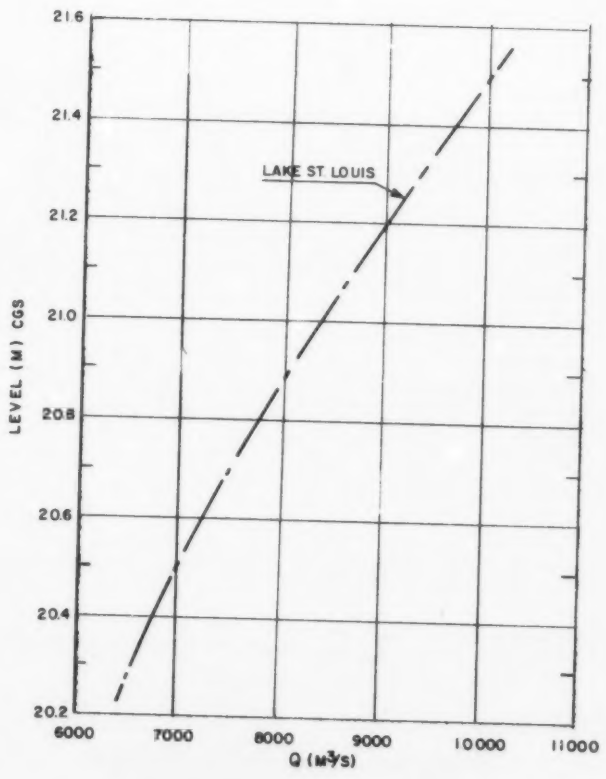


Figure.III Stage Discharge Curve for Lake St. Louis
Lock No.5 WSC Station 000092

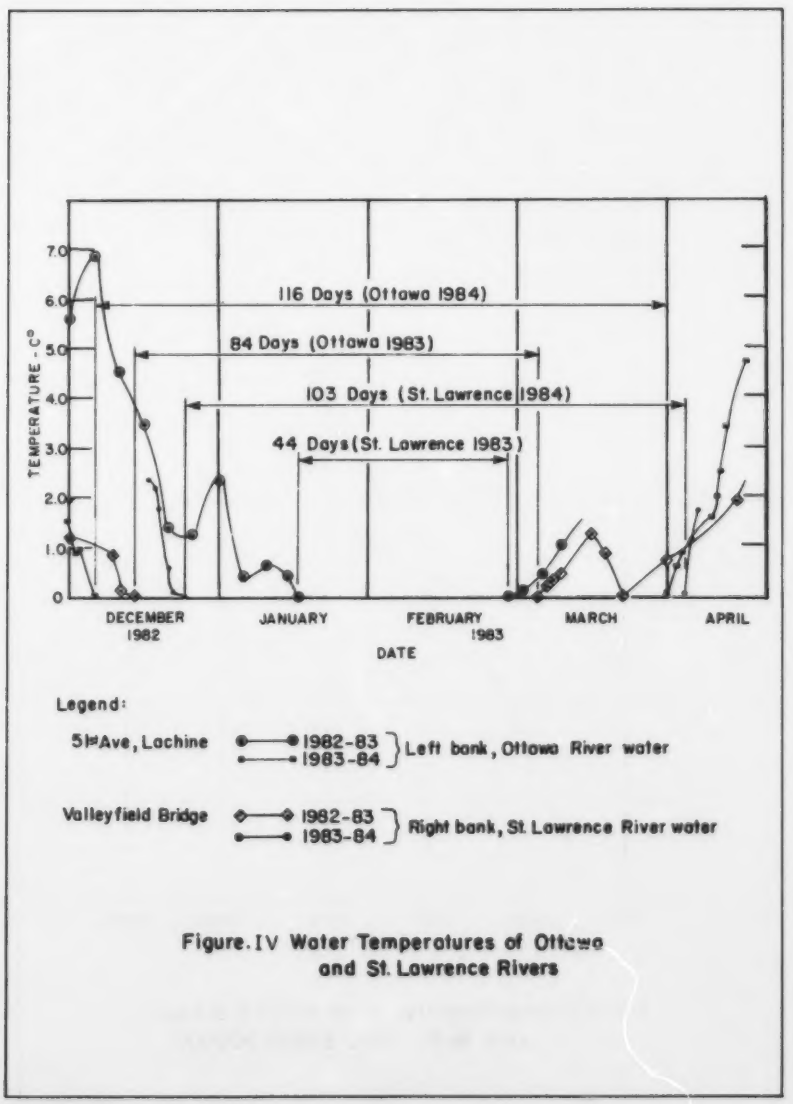


Figure. IV Water Temperatures of Ottawa and St. Lawrence Rivers

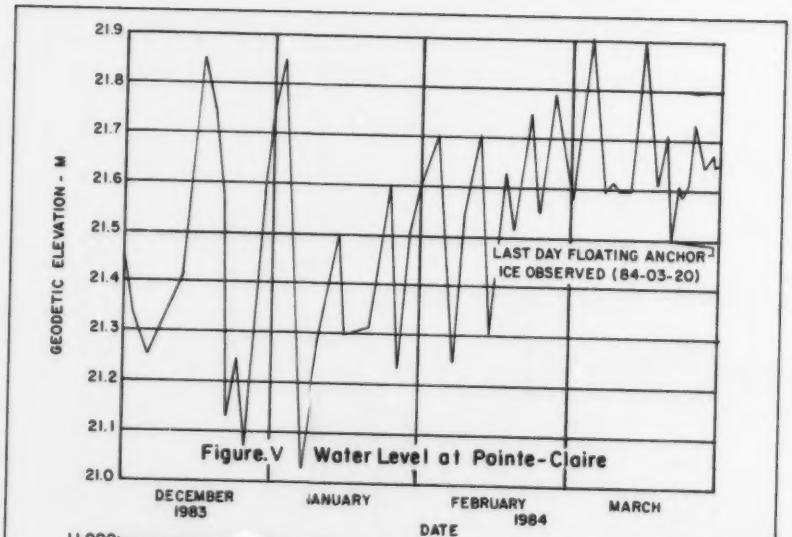


Figure.V Water Level at Pointe-Claire

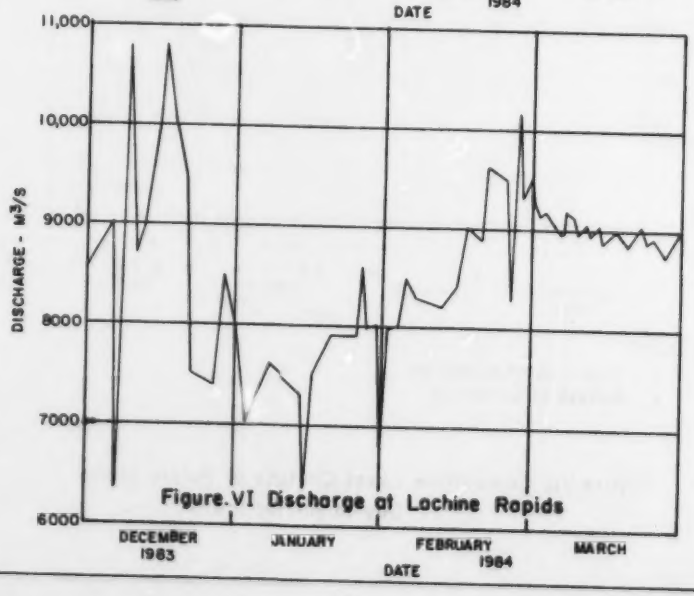
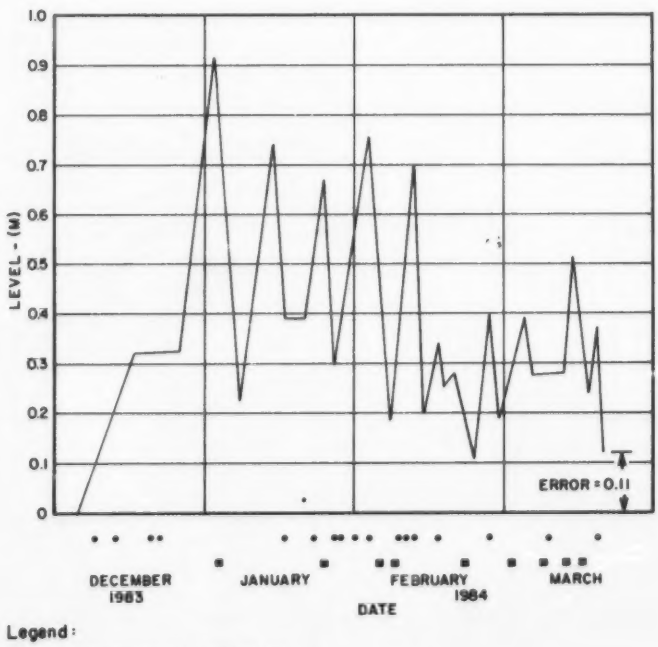


Figure.VI Discharge at Lachine Rapids



Legend:

• – Formation of Anchor Ice

▪ – Release of Anchor Ice

Figure.VII Cumulative Level Changes at Pointe Claire Due to Anchor Ice

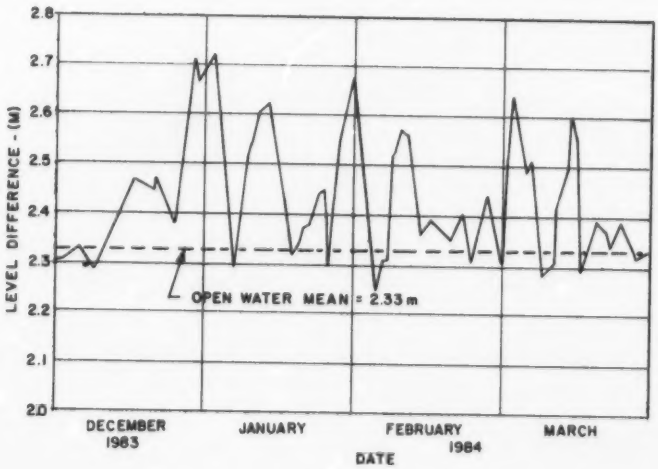


Figure. VIII Water Level Difference Between Pointe Claire and Old Aqueduct

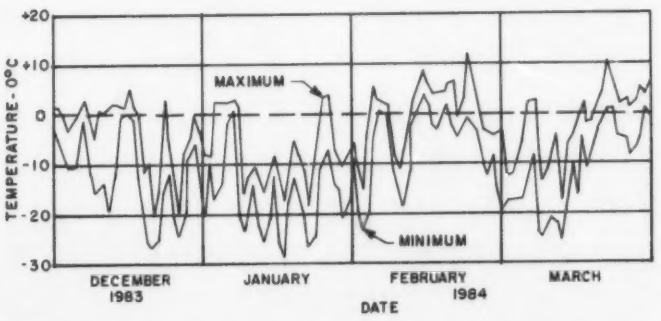


Figure. IX Maximum and Minimum Air Temperatures

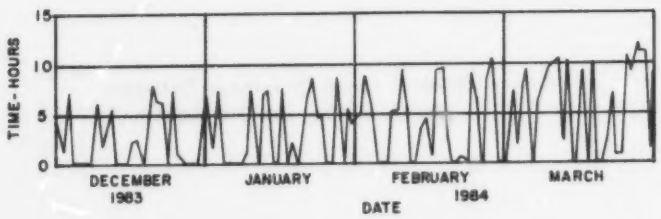


Figure. X Daily Hours of Sunshine

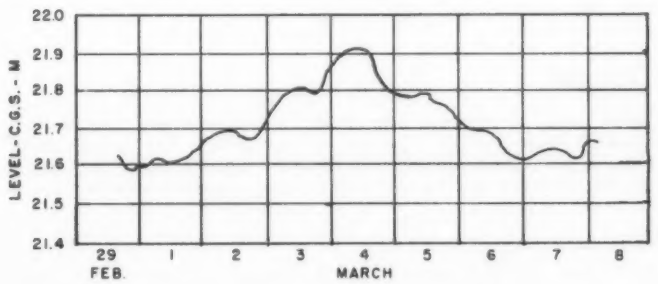


Figure. XI Levels at Pointe Claire

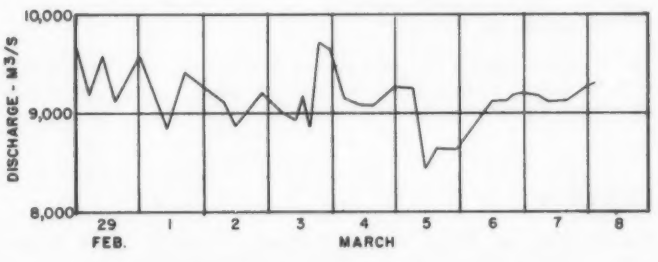


Figure. XII Discharges at Lachine Rapids

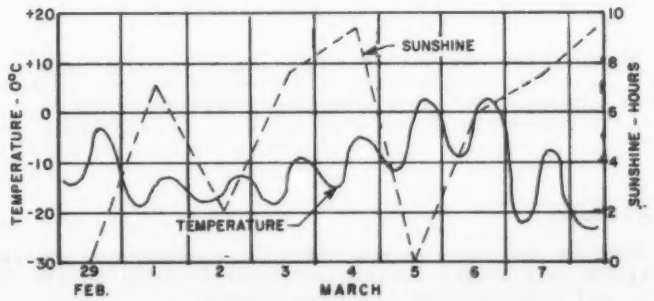


Figure. XIII Air Temperatures and Daily Sunshine

DISCUSSION

J. L. Wuebben

When you observed the frazil or anchor ice surfacing underneath the solid ice cover at what distance upstream was there an open water surface? Could you estimate the flow depth and velocity in that ice covered reach?

Reply

The solid ice cover under which the anchor ice was observed was in fact a thin ice sheet, up to 10 mm thick, and mobile. Open water upstream was therefore at variable distances depending on the arrival or generation of new ice sheets, but would generally be between 500 and 1000 m from the observation site.

The depth at the observation site varies between 1 and 3 m over the 200 m width between shore and Dorval Island. The velocity would be about 0.5 m/s.

P. R. Tremblay

Would you say that the increase in water levels caused by anchor ice production is mainly related to the increase in roughness of the river bed or to the reduction in cross-section?

Reply

My data are not sufficient to separate the two effects, and I have no information that would favour either one or the other. For the moment, I assume they are both partially responsible, but I do not know in what proportions.

D. M. Foulds

Was the ice shown in the ice cover immediately downstream of Beauharnois frazil which had been created in the Power Canal, and come through the Power House?

Reply

As near as I know, no analyses of that accumulation have been done which would identify the ice as having been freshly generated downstream of the plant or being larger, older pieces that would have come from the canal upstream of the plant. Knowing that there is a significant active frazil ice content in the canal, my own feeling is that the accumulation downstream did in fact come from the canal.

D. M. Foulds

Comment: The rate of flow reduction due to anchor ice at Niagara is 30% in 3 hours. Did you find anything similar?

Reply

My flow measurements were all for water coming into the lake, and I had no corresponding measurements at the outlet. To answer your question, I would have to do a flood routing study, knowing the lake level changes. This would be a valuable extension of the use of my field data, but it has not been done yet.

Marc Drouin

You have observed a water level increase of the order of 90 cm. Is that a maximum value and if not can you evaluate the extreme condition for the case of Lake St. Louis?

Reply

There is no reason to think that the 90 cm rise observed last winter was a maximum. The winter was about average in terms of cold weather, and had four distinct thawing periods during the anchor ice season. I can well imagine that if we were to experience a sustained longer duration cold period, like the 1980-81 winter, that the rise due to anchor ice would be greater. I have heard of rises up to 1.5 m on northern Manitoba and James Bay region rivers, so I would suggest that this same value might be possible on the St. Lawrence.

Bernard Michel

Are you using the Water Survey of Canada (WSC) gauge discharge at Lachine; and if so then how do you ascertain that there is no effect of anchor ice in the discharge itself?

Reply

The W.S.C. gauges throughout the reach were used only to give water levels. The discharge was provided by Hydro-Quebec from their mechanical model that is based on known discharges from the powerhouses at Beauharnois and Carillon. Inflows from smaller streams are taken from gauged levels, where local ice effects could introduce some error. However, these streams represent less than 5% of the total flow, so I was willing to accept this margin of error.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

IMPROVED ICE CONTROL WITH BETTER UNDERSTANDING OF THERMAL REGIME OF THE INTERNATIONAL SECTION OF ST. LAWRENCE RIVER

D. Bunte-Bisnett*, Dr. S.K. Gupta**, R.W. Penn**

ABSTRACT

Operational experience on the St. Lawrence River has indicated that a better understanding of the thermal characteristics of the International Rapids Section reach of the river would enhance the Power Entities (Ontario Hydro and New York Power Authority) capability to carry out winter operations particularly facilitating ice boom installation, facilitating the formation of an ice cover and maintaining channel capacity during the winter months.

While in the past a close monitoring of the ice phenomenon has taken place, predictions so far as to what might happen in regard to ice formation and maintaining ice stability were largely a matter of experience.

It is now believed that a major key to understanding the multiple processes that may occur is a precise awareness of the water temperature regime at all times. On the basis of this belief the Power Entities have installed an electronic thermometer and graphic recorder at the Iroquois control structure.

The purpose of this paper is to discuss the importance of knowing precisely at all times the water temperature regime before and during the ice season and relating this to the winter operational objectives. Findings from past two ice seasons will also be discussed.

It is hoped that the presentation would generate comments which will help in utilization of this data in understanding what is occurring.

*Civil and Hydraulic Engineer, New York Power Authority
**Engineer River Control, Ontario Hydro

RÉSUMÉ

L'expérience de l'exploitation des aménagements sur le fleuve St-Laurent a indiqué qu'une meilleure connaissance des caractéristiques thermiques de la section des Rapides internationaux améliorerait la capacité des entreprises d'énergie électrique (Ontario Hydro et New York Power Authority) d'effectuer les opérations hivernales, en facilitant l'installation des estacades et la formation du manteau glaciel, et en maintenant la capacité du canal pendant les mois d'hiver.

Bien que l'on ait entrepris dans le passé une surveillance étroite du phénomène glaciel, les prédictions jusqu'ici de ce qui pourrait se passer lors de la formation du manteau de glace et du maintien de la stabilité des glaces s'appuient surtout sur l'expérience.

On croit maintenant que la clef principale d'une compréhension des processus multiples possibles c'est une connaissance précise de la régime de température de l'eau à tous moments. Sur la base de cette conviction, les entreprises d'énergie électrique ont installé à l'ouvrage de contrôle d'Iroquois un thermomètre électronique et un enregistreur graphique.

Cet article a pour but de discuter l'importance de savoir précisément à tous moments la régime thermique de l'eau avant et pendant la saison des glaces et d'établir un rapport entre cette régime et les objectifs de l'exploitation hivernale. On discutera également des résultats des recherches des deux dernières saisons glacielles.

On espère que cette présentation suscitera des commentaires qui contribueront à l'utilisation de ces données pour comprendre ce qui se passe.

INTRODUCTION

Responsibility of Power Entities

Pursuant to the International Joint Commission 1952 Order of Approval for the St. Lawrence Power Development in the International Rapids Section (Cornwall-Prescott), Ontario Hydro and New York Power Authority (the Power Entities) built and have the responsibility of operating the project works, including the St. Lawrence River ice booms. These operations, which are inherent in the Regulation Plan 1958-D for the control of Lake Ontario levels, aim at protecting the rights of upstream and downstream riparian interests, navigation and downstream power producers. At the same time, the operations tend to maximize power production by maintaining winter outflows as large as feasible while minimizing the difficulties of winter power operations. In order to achieve adequate channel carrying capacity for discharging the Plan flows the establishment and maintenance of a smooth and stable ice cover is a part of the Power Entity's responsibility.

Production Process - An Overview

The processes for producing ice in a river are complex and numerous. Many theories have been put forth to describe the physics of frazil ice generation and its mode of packing to form a surface cover. A good summary of these has been given by Ashton (1978, 1979), Calkins (1982) and Tsang (1982). However, a simple overview of the ice production process will help the reader appreciate the significance of knowing the thermal regime of the International Section of St. Lawrence River for the purpose of improving ice control.

Essentially the temperature of the river must be lowered to the freezing point before any ice can start forming. This loss of heat is accomplished primarily through the processes of advection and conduction at the air-water interface, that is, the water cools due to its contact with colder air. The process of heat loss is speeded up by cold winds blowing across this interface.

To understand the mechanism involved in the production of first ice on a river, it is necessary to focus on how the moving water gives up its heat, specifically on the air-water interface. As colder air temperatures persist, the thin layer of water on the river's surface tends to lose its heat content rapidly to the colder air. Counteracting this heat loss is the process of turbulent mixing of the water. Warmer subsurface water replaces or mixes with the upper layer of colder water. Thus the lowering of the surface layer's temperature is tempered, and the eventual production of first ice is delayed. If the process carries on long enough or if there is a sudden and large drop in the air temperature, it is possible for finite parcels of surface water to reach the freezing-point and actually produce ice before being drawn into the warmer subsurface water where the ice will melt. If the cold spell is severe enough

(intensity and duration), more ice will be produced than can be drawn down and melted. An ice flow will then be established. Also, as time goes on, the whole river flow becomes colder due to the continuing interchange of cold surface water with the less and less warmer subsurface water and the river approaches the isothermal state.

If the cold spell continues, a significant volume of small ice crystals will be produced and, depending on the degree of turbulence in the river, freeze together forming larger pans of ice. These larger pieces tend to become immune to any turbulent forces existing, due to their natural buoyancy, and therefore are less apt to melt quickly. These pieces in turn freeze together, eventually forming large pans which flow downstream and pack together in a low velocity reach to form solid covers. This process continues with the upstream progression of the ice cover.

However, if warm weather develops (combination warmer air and/or sunshine), the process described above is cut short. The warmer subsurface waters tend to melt all the small crystals, and freeze-up is delayed.

The process of freeze-up on the St. Lawrence is further complicated by the fact that the source of river flow is the large, deep and relatively warm Lake Ontario; consequently, the temperature of the water entering the river can remain significantly above the freezing point (several tenths of a degree C) throughout the winter. Any tempering of a cold spell allows the river to be recharged with warmer water, and the process of heat loss from the river water must repeat itself.

FORMATION OF ICE COVER

Objective of Winter Power Operations

As indicated earlier, promotion of a smooth and stable ice cover over the International Section of the St. Lawrence River is carried out to maintain adequate channel capacity to pass flows prescribed by Plan 1958-D and hence to control the levels of Lake Ontario within prescribed limits as well as to minimize the head loss at the Power Dam. It is also desirable to optimize power output by maintaining as high a flow as possible while the cover is forming and to be able to increase the flow after it has formed without undue risk of having the cover collapse.

Role of Channel Enlargements and Ice Booms

To aid the formation of a smooth and stable ice cover during winter the natural channels between the powerhouses and Sparrowhawk Point (Figure 1) were enlarged by excavation so that the river velocities average 0.686 m/s with a steady flow of 6 230 m^3/s. However, such enlargements further upstream were found to be very uneconomical and hence not made.

Consequently, average velocities of up to 0.914 m/s can be anticipated during ice cover formation period between Sparrowhawk Point and Prescott.

During the first winter of operation of the Power Project, in 1959, a severe ice jam occurred near Cardinal, Ontario, as a result of high winds. The jam reduced St. Lawrence River flows and levels resulting in low water problems for riparian interests, severe power production losses and induced higher Lake Ontario levels than what would have resulted if the prescribed regulated flows had been released. In order to avoid recurrence of such ice jamming the Power Entities installed six ice booms in the Prescott-Galop Island reach. These ice booms curtail the quantity of ice passing Prescott and restrict the movement of ice between Prescott and Cardinal, thus helping in the formation of a smooth and stable ice cover.

Significance of Awareness of Water Temperature Regime
for Critical Decisions During Winter Operations

Winter operations are influenced by the hydrological conditions of the Great Lakes basin and the meteorologic, hydraulic and physical conditions of the International Rapids Section. The winter operations are divided into two phases:

(i) Initial freeze-up and ice cover stabilization phase.

(ii) Ice cover maintenance (mid-winter) and "melt-out" phase.

During all phases of operation, continuous monitoring occurs of the factors which influence the rate and quality of ice formation and the degree of stabilization achieved. A description of winter operations procedure has been given by Messrs. Wigle, Bartholomew and Lawrie (1981). Detailed information regarding the state of ice regime is particularly crucial while making decisions for closing the ice booms at the start of ice season and also while attempting to maximize flow releases from Lake Ontario during the ice season. It is believed that knowledge of water temperature regime would significantly aid the Power Entities in making critical decisions during winter operations.

Initial Freeze-Up

(a) Ice Boom Closure

It is normal practice to initiate ice boom closure procedures when the temperature at the Power Dam falls below 0.6°C or following passage of the last commercial vessel through the boom area, whichever occurs first. This normally results in satisfactory winter operations However, if the "freeze-up period" arrives early and/or navigation continues into the critical ice-forming period preventing the closure of the booms at the proper time, the ice cover which would otherwise consolidate gets broken up, and

formation of a smooth and stable ice cover is rendered difficult as happened in the winter of 1976-77.

If a section of ice boom is left open for the ships to pass, large quantities of ice may move downstream and may cause jamming in critical areas. Also, if the weather conditions remain conducive to significant quantity of ice running through boom areas, final closure of the booms becomes extremely difficult. Under these circumstances the Power Entities are faced with the question "Must the booms be closed today, cutting off the shipping or can the booms be left open for another night?" At such times, an awareness of water temperature regime in conjunction with short term weather forecast would enable the Power Entities to better assess the expected ice regime in the boom areas and make rational decision regarding closing of the booms.

(b) Flow Regulation

During initial freeze-up, to promote the formation and upstream progression of a smooth ice cover in certain critical areas it is necessary to reduce velocity by a reduction in discharge. The idea being that in areas of high velocity (such as Ogden Island channels), velocity reduction will allow the pans of ice reaching the ice front to butt against it rather than being drawn under the cover increasing head loss. Traditionally, aerial and ground surveillance of the status of the ice cover and ice runs in the International Rapids Section Reach, monitoring of real time meteorological conditions and short term meteorological forecast have been used as a guide in making flow change decisions. It is now believed that knowledge of water temperature regime of the river reach would further supplement the information on which the Power Entities can base their decision regarding flow changes, by providing better insight into ice producing processes.

Mid-Winter Period

(a) Ice Cover Maintenance

When a solid ice cover has formed, a certain structural integrity of the ice exists which resists the stresses placed upon the cover by river flow on the underside and wind on the surface. The structural integrity of ice is a direct function of air and water temperatures. If either or both are greater than 0°C long enough, the strength of the ice is weakened (the molecular structure of the crystal tends to revert to the molecular structure of the liquid). Such a weakened ice cover is less able to resist the stresses of flow and wind. It thus appears that water temperature could be a key component in monitoring the structural integrity of the cover.

During mid-winter the Regulatory Policy can dictate high discharges from Lake Ontario which the cover may not sustain and the Power Entities have to determine what maximum discharge can be tolerated by the ice cover under anticipated meteorological condition. It is believed that monitoring of water temperature (diurnal change), in addition to reliable weather forecast, would enable the Power Entities to place more confidence in a particular flow regulation strategy, that is based on fact rather than intuition.

On the basis of this belief the Power Entities have installed an electronic thermometer and graphic recorder at the Iroquois Control Structure which is briefly described in next subsection.

THERMOGRAPH SYSTEM

A digital thermometer was built by the Hydraulic Flow Systems Lab of Ontario Hydro to certain specifications the major ones being:

1. to operate with an accuracy, repeatability, resolution, and stability of better than $\pm$0.01°C (at freezing point problem temperature)

2. to provide an accurate measure of temperature in the field, at about the freezing point of water where the instrument frequently would be subjected to ambient temperatures down to -20°C.

The thermograph system consists of a linear composite thermister with a probe range of -5°C to 39.99°C connected to an amplifier and a continuous graphic record (Figure 2).

The thermograph system is installed on the Iroquois Dam in the centre of flow at approximately Pier 23 of 32 piers. Normally the probe is in roughly 10 feet of water. In order to obtain water temperature a well mixed zone, the position of the probe is near the edge of the pier.

The installation was completed in December of 1980 and the first two ice seasons were a teething period for the system. Water temperatures are graphically recorded to provide a complete record of water temperatures throughout the winter season.

Calibration

Confidence exists in the data gathered in the winters of 1982-83 and 1983-84. Procedures were developed to ensure a weekly calibration of the thermograph system.

A separate portable thermometer again accurate to $\pm$.01°C is used to calibrate the in-situ thermograph system. Weekly the portable unit is checked for accuracy using an ice bath to ensure reading 0.00°C. A comparison is then made of water temperatures from the calibrating unit

and the in-situ unit, due to the difficulty in doing an ice bath on the in-situ unit. Discrepancies are noted between the two and the recorded data altered if an error exists.

Data Available

Water temperature information is available, see Figure 1 for locations, from:

(a) Iroquois Control Dam - continuous record (± .01°C)
(b) Saunders-Moses GS - 8:00 am reading (± .01°C)
(c) Kingston PUC - 8:00 am reading (± .10°C)

FINDINGS TO DATE

Water temperature information provided by the thermograph system combined with operational experience over the past two winters has provided some initial findings.

Freezing-Degree Days

(a) The first area of concern during a winter freeze-up is the prediction of when freeze-up will occur. The necessity being to determine timing of the ice boom closure and forewarn of the possibility of the navigation and ice season overlapping.

It was felt that freezing-degree days may be a key to the prediction of the beginning of ice formation as has been reported by Greene (1983). As an initial step, freezing-degree days have been summed from December 1st of each year for the past 13 years and have been compared against the number of days after December 1st when the ice formation process began. (Figure 3.)

At best this provides a ballpark estimate of ice formation. In the future we hope to reduce the scatter of these findings by taking into account windspeed, perhaps using windchill as a factor and creating windchill-freezing degree-days.

(b) Zero Degree Line

As discussed earlier a unique feature of the thermal regime on the International Rapids section is the relatively warm heat source, Lake Ontario, upstream providing warm water throughout the winter.

As the St. Lawrence River flows north away from Lake Ontario, water temperatures in the river begin to cool until at some point in time they reach 0.00°C or lower and ice begins to form.

What can be observed is the upstream movement of a "zero-degree line" as water temperatures both in the river and at Lake Ontario

cool due to air temperature, winds, radiation, etc. Observations during the winter of 1982-83 show this upstream movement of the zero-degree line (Figure 4).

We have observed that during the initial ice formation period, when the ice cover remains fairly fragile and thin, in order for the cover to not recede it is necessary for the zero-degree line to be upstream of the ice front.

Such a phenomena was observed during the winter of 1982-1983 when a cover formed but lasted for a short period of time during mid-January and again in early to mid-February as seen in Figure 4.

It is quite logical to expect that if there is too much heat available in Lake Ontario for the normal cold winter, a permanent cover will not form but will be thermally eroded. A good indicator of such a phenomena appears to be the Iroquois and Kingston water temperatures to determine the temperature differential between stations.

Initial results indicate that if water temperatures in Lake Ontario are higher than .6 to .7°C a permanent ice cover will not form.

(c) Windchill

Another factor that is now being examined in the ice formation process is the effect of windspeed upon water temperature depression.

Windspeed has been combined with air temperature to provide a windchill factor or the apparent temperature. Although not wholly scientific windchill appears to provide a simple indicator when looking at the depression of water temperature.

Figure 5 (a) shows water temperature over a 96-hour period on February 27 to March 1, 1984. When compared against air temperature Figure 5 (b) for the same period no correlation exists between the dropping water temperature and constant air temperature. With windchill factored in, the trend shown by the windchill closely follows the trend in water temperatures.

It is hoped that in the future further studies will find a good correlation between windchill and depression of water temperature. Thus, a good forecast of air speed and expected air temperature would help the Power Entities in the prediction of the timing of ice runs for possible flow decisions.

(d) Surface Ice Runs

Although the general belief is that for an ice run to occur, the river cross-section should be isothermal at 0.00°C or supercooled.

Direct observations indicate that surface runs are occurring with water temperatures greater than 0.00°C measured at the thermograph (10 feet under water).

The writers have not yet been able to find a full explanation for this phenomenon although the explanation put forth by Tsang (1982) for turbulence and mixing in deep rivers may be applicable.

In the future, we plan to measure temperature profiles at the Thermograph site to determine if and why surface runs are occurring and if the river is isothermal.

Future Studies

With the weekly calibration program in place, it is felt the Thermograph System is working to specification (accuracy of $\pm$ 0.01°C) and providing water temperature information that is fairly representative of the river at Iroquois Dam.

Now that we have confidence in the data the Thermograph system is providing we intend on continuing to investigate correlations between water temperature and the other factors affecting ice production and cover formation. The hope being the Power Entities will have a tool to substantiate their intuitive decisions.

The authors do look forward to suggestions that may be helpful in utilizing the water temperature information for winter operations on the St. Lawrence River.

List of References

Ashton G.D., (1978), River Ice, Annual Review, Fluid Mechanics, Vol. 10, pp 369 to 392

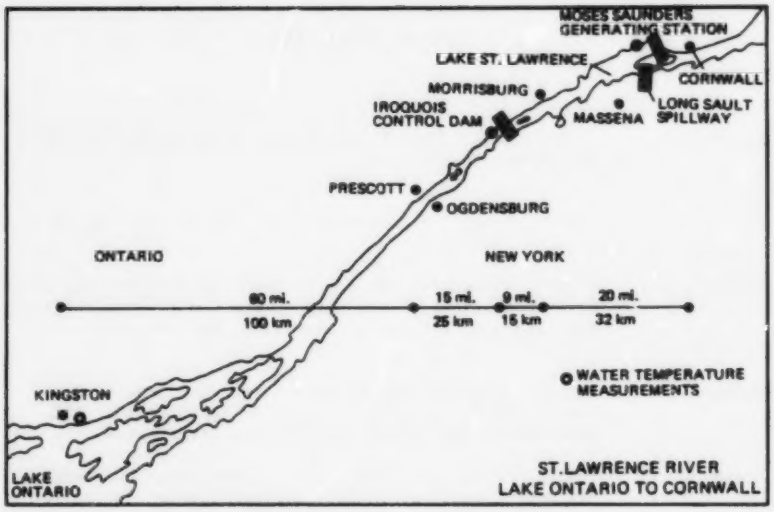
Ashton, G.D., (1979), River Ice, American Scientist Vol. 67, pp 38 to 45

Calkins, D.J., (1982), Hydraulics, Mechanics and Heat Transfer for Winter River Conditions, Class Notes: Ice Engineering for Rivers and Lakes, University of Wisconsin, Madison, WI

Greene, G.M., (1983), Forecasting Ice Cover Freeze-Up, Growth, and Breakup on the St. Marys River, National Oceanic and Atmospheric Administration Technical Memorandum

Tsang, G., (1982), Frazil and Anchor Ice - A Monograph, National Research Council of Canada, Subcommittee on Hydraulics of Ice Covered Rivers, Ottawa, Ontario, Canada

Wigle, T.E., Bartholomew, J., and Lawrie, C.J.R. (1981) - Winter Operations, International Rapids Section of the St. Lawrence River, International Symposium on Ice, IAHR Quebec City, Canada


MOSES SAUNDERS
GENERATING STATION
LAKE ST. LAWRENCE
CORNWALL
MORRISBURG
IROQUOIS
CONTROL DAM
MASSENA
LONG SAULT
SPILLWAY
PRESCOTT
OGDENSBURG
ONTARIO
NEW YORK
60 mi.
100 km
15 mi.
25 km
9 mi.
15 km
20 mi.
32 km
WATER TEMPERATURE
MEASUREMENTS
KINGSTON
LAKE
ONTARIO
ST.LAWRENCE RIVER
LAKE ONTARIO TO CORNWALL

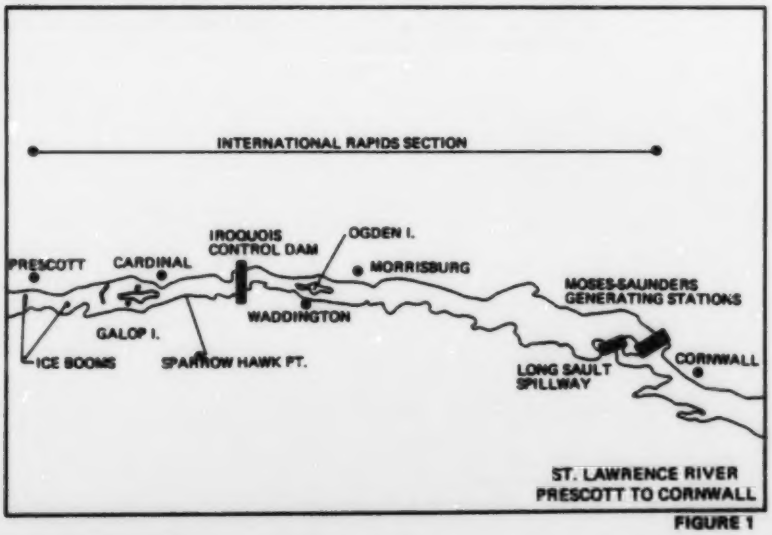
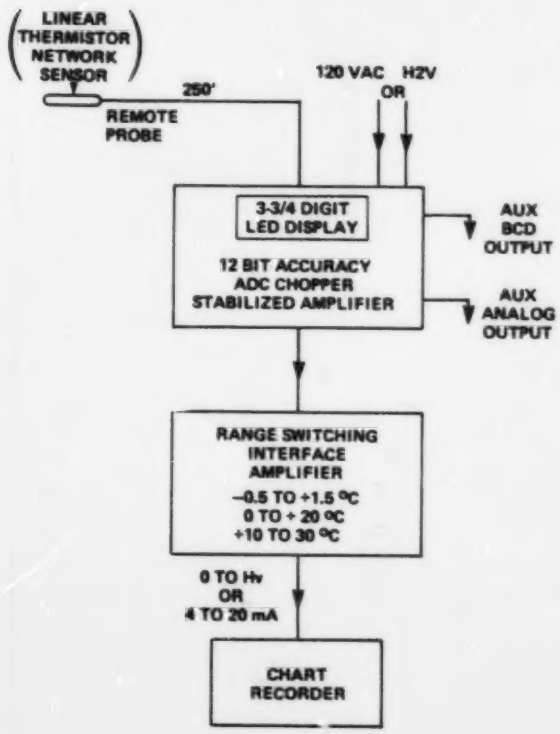

INTERNATIONAL RAPIDS SECTION
IROQUOIS
CONTROL DAM
OGDEN I.
PRESCOTT
CARDINAL
MORRISBURG
MOSES-SAUNDERS
GENERATING STATIONS
WADDINGTON
GALOP I.
ICE BOOMS
SPARROW HAWK PT.
LONG SAULT
SPILLWAY
CORNWALL
ST. LAWRENCE RIVER
PRESCOTT TO CORNWALL

FIGURE 1

FSL ELECTRONIC THERMOGRAPH SYSTEM



POWER REQUIREMENTS

120 VAC 0.5A OR
12 VDC OR
INTERNAL BATTERIES

ENVIRONMENTAL RANGE +20 TO +35°C FOR ±0.01°C ACCURACY

FIGURE 2

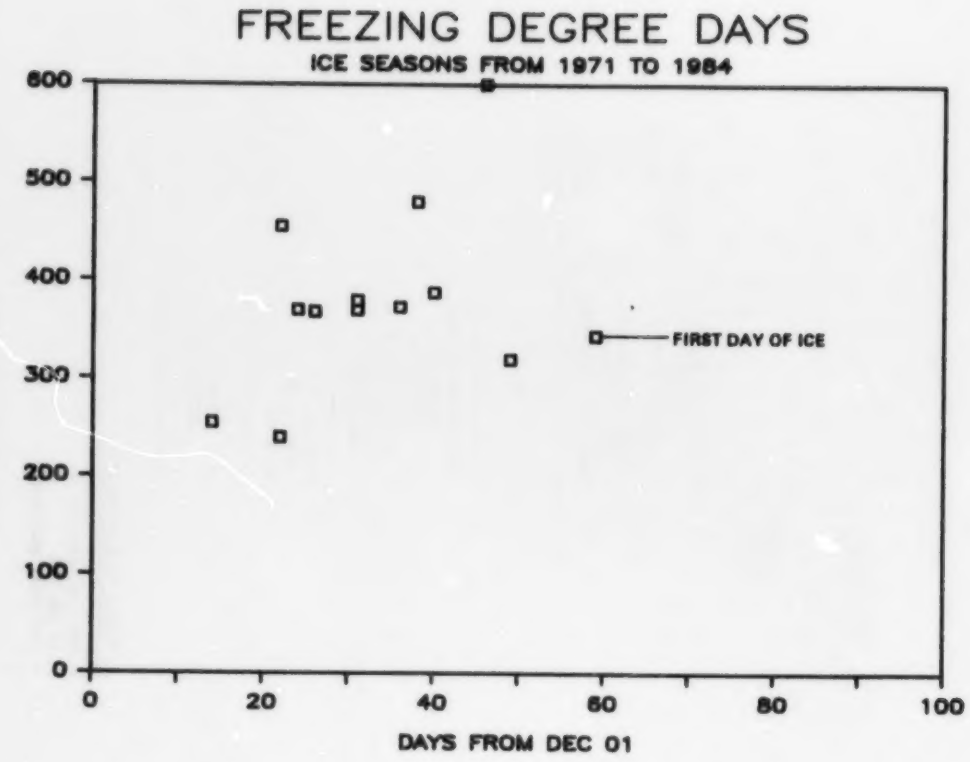

FREEZING DEGREE DAYS
ICE SEASONS FROM 1971 TO 1984
Σ FREEZING DEG. DAYS (F)
600
500
400
300
200
100
0
0
20
40
60
80
100
DAYS FROM DEC 01
FIRST DAY OF ICE

FIGURE 3

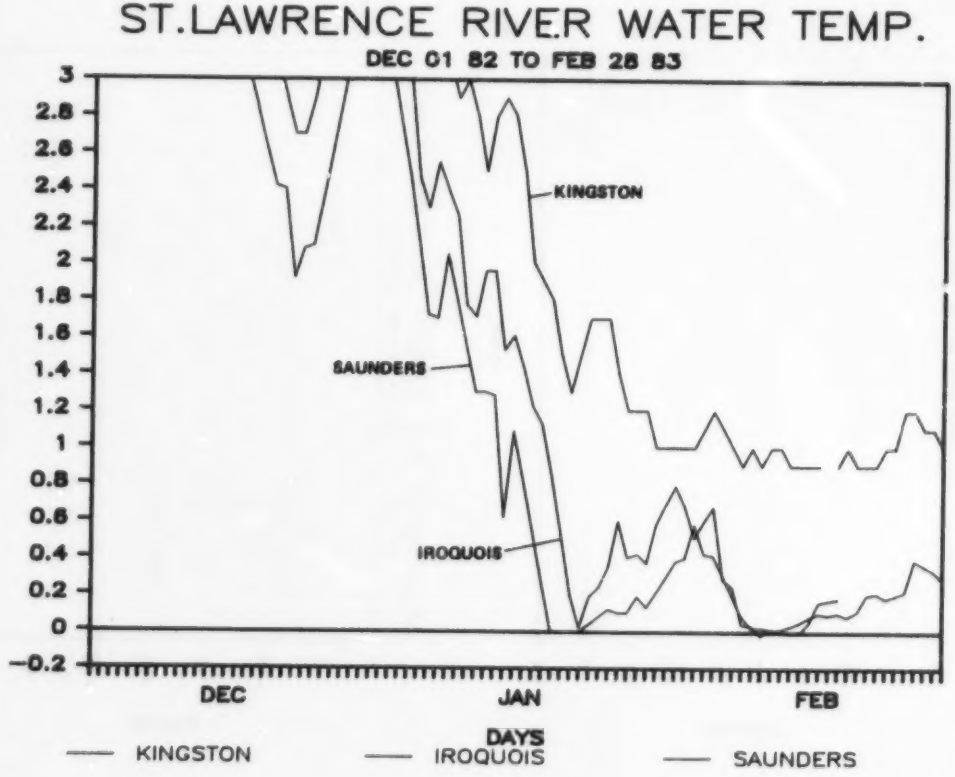

ST.LAWRENCE RIVER WATER TEMP.
DEC 01 82 TO FEB 28 83
DEGREES C
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
KINGSTON
SAUNDERS
IROQUOIS
DEC
JAN
FEB
DAYS
KINGSTON
IROQUOIS
SAUNDERS

FIGURE 4

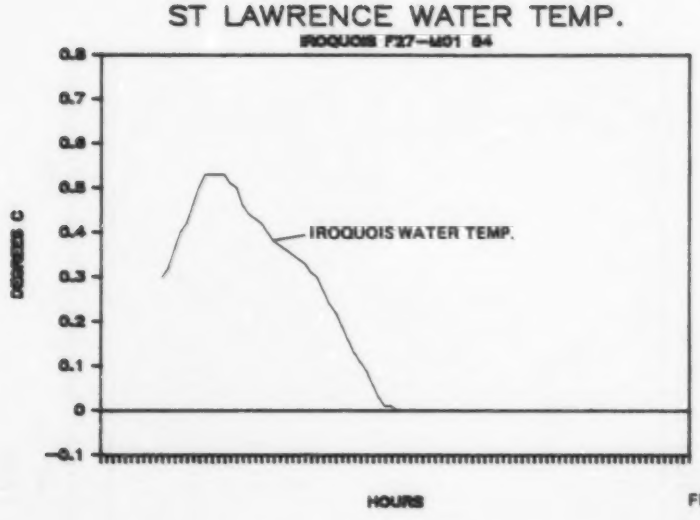


FIGURE 5 (a)

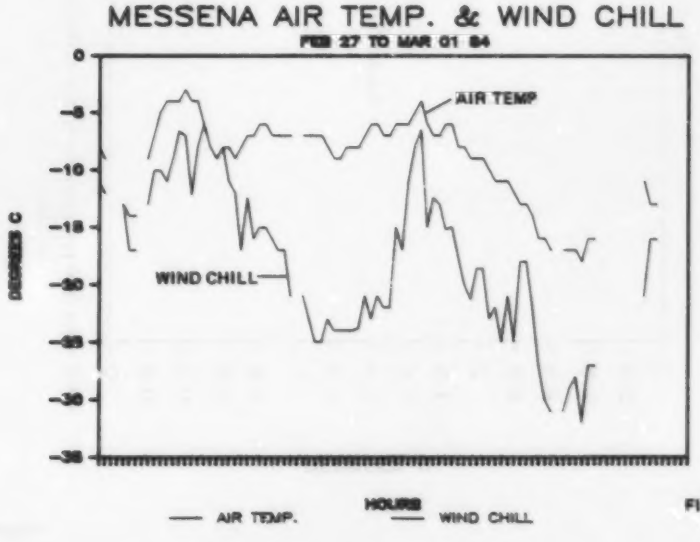


FIGURE 5 (b)

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU BRUNSWICK

INTERACTION BETWEEN SURFACE WAVES AND A DENSELY PACKED BROKEN ICE COVER

by E.P. Foltyn[1] and C.D. Ponce-Campos[2]

ABSTRACT

The interaction of surface waves and a broken ice cover is of interest in river ice hydraulics because surface waves are generated in navigable channels with the passage of every vessel as well as during storms. These waves can cause damage and erosion to riverside structures and fisheries. A broken ice cover affects an approaching wave train by reflecting a fraction of the incident wave energy and by damping the transmitting energy exponentially as a function of distance from the leading edge of the ice cover. Experiments were performed in a 40x2-ft laboratory tank and an ice cover was simulated with flat low density polyethylene. The water displacement was measured with a series of electrical resistance wire probes located at convenient intervals along the test section starting at the leading edge of the ice cover. The wave damping rate was found to depend on the size and concentration of the ice cover with relation to the characteristics of the surface waves. A functional relationship between the size and thickness of an individual ice floe and the wave number was established. Laboratory testing verified a rapid damping of relatively short waves and a very slow damping for long waves. Results of these tests include correlated measurements of reflection and transmission coefficients.

[1] Water Systems Engineer, Onondaga County Water Authority, Syracuse, N.Y. 13211.

[2] Assistant Professor, Clarkson University, Potsdam, N.Y. 13676.

INTERACTION ENTRE DES ONDES DE SURFACE ET UN AMAS COMPACT DE GLACES FLOTTANTES

L'interaction d'ondes de surface et d'un amas de glaces flottantes présente un intérêt pour l'hydrolique de la glace sur les rivieres, du fait que les ondes se produisent sur les cours d'eau navigables chaque fois que passe un bateau et pendant les orages. Ces ondes peuvent éroder et endommager les constructions et pêcheries situées sur le rivage. Or las présence d'un amas de glaces flottantes affecte un train d'ondes qui se dirige vers lui. L'amas de glaces réfléchit une fraction de l'énergie de l'onde incidente et amortit l'énergie de transmission exponentiellement en fonction de la distance du bord le plus avancé de l'amas de glaces. Des expériences ont été faites en laboratoire, à l'aide d'une cuve d'eau dans laquelle on a simulé un amas de glaces avec du polyéthyléne plat à densité basse. Le déplacement de l'eau a été mesuré par des sondes de fil métallique placées à des intervalles commodes le long de las section choisie pour le test, section partant du bord avancé de l'amas de glaces. On s'est basé, pour mesurer, sur le principe de la résistance électrique. On a trouvé que le taux d'amortissement des ondes dépend de las taille et du degré de concentration de glaces par rapport aux caractéristiques des ondes de surface. Une relation fonctionelle entre la taille et l'épaisseur d'un glaçon particulier et le nombre d'ondes a été établie. Les tests effectués en labortoire ont confirmé la rapidité de l'amortissement des ondes relativement courtes et la lenteur extrême de l'amortissement des ondes longues. Entre autres résultats, ces tests ont permis de mettre en corrélation les mesures des coefficients de réflection et de transmission.

INTRODUCTION

In recent years there has been a surge in interest to understand the phenomena of ice-wave interaction. This has come about because of interest in maintaining shipping through a normally ice covered waterway such as the St. Lawrence Seaway and because of the damage to public and private property that may be accrued seasonally. In addition to the problems of ice covered rivers, the exploration and production of petroleum products in arctic environments such as the Beaufort Sea, the North Sea, and the North Atlantic have created a need for accurate modeling of wave-ice interaction phenomena.

As a wave train approaches an icefield, part of the wave energy is reflected while the remaining is transmitted through the ice pack. The damping of the transmitted energy will be shown to be a function of penetration into the ice pack and the size and concentration of the ice floes with relation to the characteristics of the incoming waves.

This article summarizes a theoretical analysis and experimental observations of wave-ice interaction performed at the Hydraulics Laboratory at Clarkson University.

PROBLEM FORMULATION AND GENERAL SOLUTION

The Navier-Stokes and the continuity equations are the governing equations in water wave mechanics. With the incompressible and irrotational flow assumptions, it can be shown that the governing equations reduce to the Laplacian of a velocity potential, which is a special case of the continuity equation.

The potential function in two-dimensional flow is defined as:

$$u = \frac{\partial \phi}{\partial x} \quad ; \quad v = \frac{\partial \phi}{\partial y}$$

And the continuity equation as:

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0$$

Hence:

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0 \qquad \text{(1a)}$$

which can be written as:

$$\nabla^2\phi = 0 \qquad (1b)$$

Weitz and Keller (1950) developed the boundary conditions necessary to solve equation 1 and obtained a general solution of the potential function using the following assumptions:

a) Two-dimensional plane wave

b) Linearized, small amplitude wave theory in water of finite depth

c) The potential function is harmonic with time, i.e.,

$$\phi(x,y,t) = e^{i\sigma t}\,\phi(x,y)$$

d) The bottom is horizontal and flat.

e) Negligible submergence of the ice.

At the point that a plane wave approaching from an open water area meets with edge of a broken ice pack, a fraction of the wave energy is reflected from the ice pack and the remaining is transmitted into it, as shown in the definition sketch in Fig. 1.

For the case of waves traveling in the open water a well known procedure to establish the boundary conditions is based on a fixed and impermeable bottom boundary. Hence, at the bottom, where y=0, let the surface of the boundary be Y = y and remain a constant. This boundary is at the same time a streamline and its material derivative must be zero.

$$\frac{DY}{Dt} = \frac{\partial y}{\partial t} + u\,\frac{\partial y}{\partial x} + v\,\frac{\partial y}{\partial y} = 0 \qquad (2a)$$

This verifies the no flow condition across the solid boundary:

$$v = \frac{\partial \phi}{\partial y} = 0 \qquad (2b)$$

Similarly at the free surface Y = y-η, and the kinematic condition here is that the velocity v equals the rate of change of water surface elevation. Hence:

$$v = \frac{\partial \phi}{\partial y}\Big|_Y = \frac{\partial \eta}{\partial t} \tag{3}$$

Because the location of the free surface η is a variable, we introduce an additional condition to fix the location of . By utilizing the linearized Bernoulli equation and allowing the pressure term, p, to be atmospheric we arrive at

$$\eta = -\frac{1}{g}\frac{\partial \phi}{\partial t} \tag{4}$$

By differentiating Eq. 4 with respect to time, substituting into Eq. 3 and utilizing the assumption that the potential is periodic in time we obtain

$$\frac{\partial \phi}{\partial y} = \frac{\sigma^2}{g}\phi \tag{5}$$

For the case of ice covered waters, an analogous procedure yields:

$$\frac{\partial \phi}{\partial y}\Big|_{Y=0} = 0 \tag{6}$$

for the kinematic boundary condition at the bottom.

At the ice-water interface one has to account for the effect of the floating ice particles. On the assumption that the length of the ice particle is of the order of the wave length, or less, the elastic properties of the ice become negligible. This is termed the discrete-inelastic condition. In addition interparticle friction and submergence is considered negligible, and the response of the particle instantaneous. The Equation of Motion of the particle and Bernoulli's Equation can be coupled to obtain:

$$\frac{\partial \phi}{\partial y} = \left(\frac{\rho\sigma^2}{\rho g - \rho_i \theta \sigma^2}\right)\phi \tag{7}$$

The force balance used in Eq. 7 is shown on Fig. 2.

We can now look for a separable solution to Laplace's Equation in the form of a periodic wave traveling in the x-direction.

$$\phi(x,y) = X(x)\ Y(y)$$

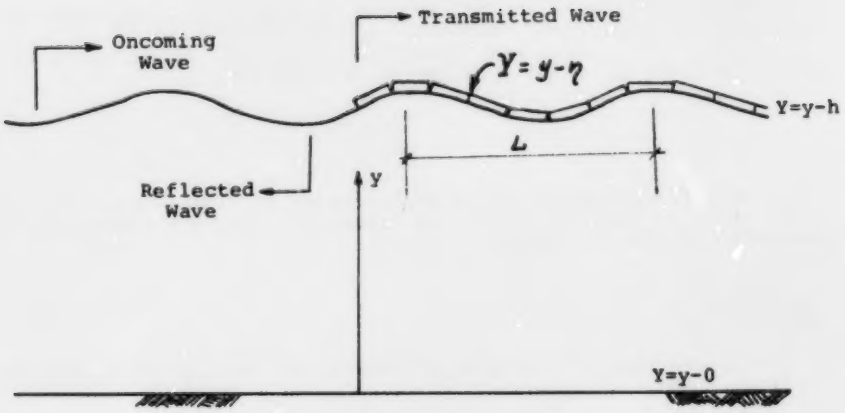


Fig. 1. Definition Sketch

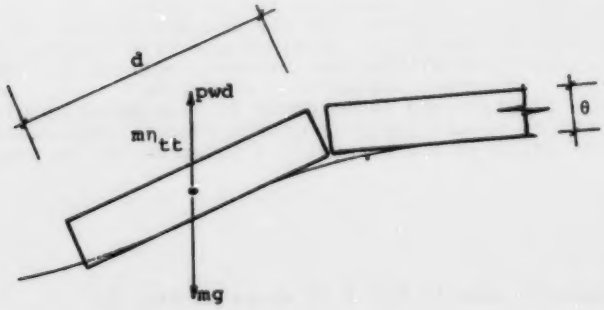


Fig. 2. Forces on Ice Particle

Substitution into Eq. 1 yields

$$\frac{X''}{X} = -\frac{Y''}{Y} = -k^2$$

which has solutions of the form

$$X = ae^{ikx} + be^{-ikx} \tag{8a}$$

and

$$Y = A \cosh kx + b \sinh kx \tag{8b}$$

The kinematic boundary condition given by Eq. 6 dictates:

$$Ak \sinh(k(0)) + B\, k \cosh(k(0)) = 0$$

Therefore B=0. Equation 5 or 7 can be written as

$$\frac{\partial \phi}{\partial y} = \beta\phi \tag{9}$$

where the constant β is defined as

$$\beta = \frac{\rho\sigma^2}{\rho g - \rho_i \theta \sigma^2}$$

when the ice thickness θ goes to zero, Eq. 7 becomes Eq. 5.

When β is greater than zero, Eq. 9 has two real roots given by $\pm k_o$, which can be obtained from

$$k_o \sinh(k_o h) = \beta \cosh(k_o h) \tag{10a}$$

In addition, all β have an infinite number of imaginary roots given by

$$(ik_n)\sinh(ik_n h) = \beta \cosh(ik_n h) \tag{10b}$$

which can be reduced to:

$$-k_n \sin(k_n h) = \beta \cos(k_n h) \tag{10c}$$

The eigenvalues of these functions are shown in Fig. 3.

In general, the delineation between the propagating wave and a decaying wave depends on the value of β. Since the magnitude and sign of β depends upon the relative value of the terms of the denominator, it follows that

$$\beta > 0 \qquad \rho g - \rho_i \theta \sigma^2 > 0$$

where $\beta > 0$ imples both wave propagation and decay. By contrast

$$\beta < 0 \qquad \rho g - \rho_i \theta \sigma^2 < 0$$

implies wave decay only.

With $\sigma = 2\pi/T$ and the ratio $\rho_i/\rho = 0.92$ we obtain for a decaying wave, in SI units:

$$\frac{\theta}{T^2} > 0.27 \frac{m}{s^2} \tag{11}$$

The real eigenvalues k_o correspond to the wave number $k_o = 2\pi/L$.

The dispersive relationship is obtained by substituting this solution into Eq. 10, as has the form

$$L = \left(\frac{gT^2}{2\pi} - \rho_i \theta 2\pi\right) \tanh \frac{2\pi h}{L} \tag{12}$$

Combining Eq. 11 and Eq. 12, one can show that for any particular wave field the criteria between a decaying and non-decaying wave field will be given by

$$\frac{\theta}{L} > \text{const.} \tag{13}$$

The constant in Eq. 13 results from the physical characteristics of the problem and must be computed in each case.

The general solution for this wave field is then:

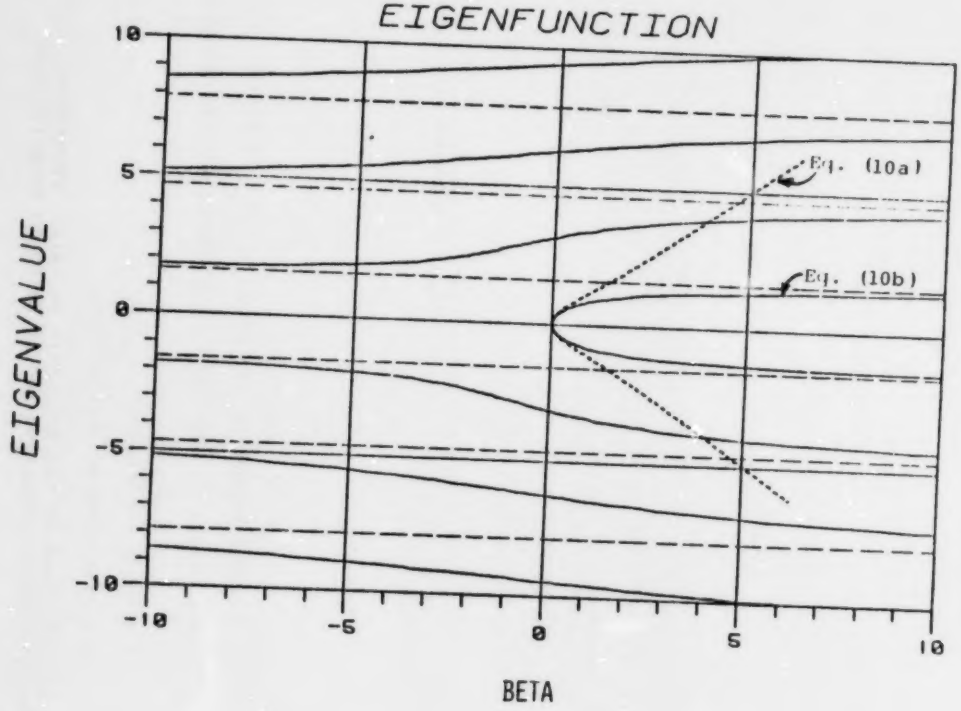


Figure 3. EIGENVALUES FOR EQUATIONS (9) and (10)

$$\underset{(x>0)}{\phi(x,y,t)} = \sum_{n=1}^{\infty} a_n e^{-(k_n x + i\sigma t)} \cos(k_n y) + a_o \cosh(k_o y) e^{i(k_o x - \sigma t)} \qquad (14a)$$

$$\underset{(x<0)}{\phi(x,y,t)} = \sum_{n=1}^{\infty} b_n e^{(\lambda_n x - i\sigma t)} \cos(\lambda_n y)$$

$$+ b_o \cosh(\lambda_o y)[e^{i(\lambda_o x - \sigma t)} + e^{-i(\lambda_o x + \sigma t)}] \qquad (14b)$$

Equation 14a is the potential function for an ice covered wave field while Eq. 14b is the potential accounting for an incoming and reflected wave in open water.

A dimensional analysis is helpful here to determine any additional relationships. Since

$$\phi = \phi(\theta, d, H, h, L, g, \rho, \rho_i, T, x, y \ldots)$$

The first term in Eq. 14a shows that the decay of the wave amplitude as it travels into the ice pack is exponential. Hence, it is reasonable to state:

$$H = H_o e^{-(\delta + \gamma x)} \qquad (15)$$

where:

H is the wave height at any point in the ice field

H_o is the height of the combined incoming and reflected wave

γ is a decay coefficient

δ is a transmission coefficient,

The meaningfulness of the d/L ratio is also emphasized by the discrete-inelastic condition and it is to no surprise that

$$\gamma = \gamma\left(\frac{d}{L}\right) \quad ; \quad \delta = \delta\left(\frac{d}{L}\right)$$

EXPERIMENTAL PROCEDURE

Experimental verification of the preceeding analysis was carried out in the 13.7 m (40-ft) long tilting open channel flume in the Hydraulics Laboratory of the Department of Civil and Environmental Engineering at Clarkson University. The channel is 0.6 m (2-ft) wide and square in cross-section. The flume is constructed of a structural steel frame with plexiglass sidewalls. The inlet side of the flume was fitted with a paddle type wave generator with a continuously variable period of 0.3 to 2.0 seconds. At the far end of the tank, a beach with a 7 horizontal slope and 2.4 m (8-ft) long was used to absorb the wave energy. The beach was constructed of perforated sheet metal and covered with a 10 cm (4-in) thick layer of filtering material to ensure good wave damping. A 5 cm (2-in) thick sheet of the same material was also used directly in front of the wave paddle to smooth out the wave profile created by turbulence at the paddle.

Wave heights were measured using a resistance wire probe. This probe consisted of 4 lengths of high resistance (98 ohms/m) thermocouple wire looped on the outside of a 0.6 cm o.d. (1/4-in) plexiglass tube. As the water level at the probe changed a liner change in resistance was observed. This probe was one of the resistors in a 4-legged a.c. bridge circuit. This circuit was driven by a Hewlett-Packard carrier amplifier and the resulting change in voltage was monitored by a Hewlett-Packard chart recorder. The gage circuit diagram is shown in Fig. 4.

The ice cover was modeled by utilizing a 1.3 cm (1/2-in) thick sheet of low density polyethylene cut into square blocks slightly smaller than the width of the tank. The blocks were kept in position by a rubber band hinge.

To determine the wave attenuation rate, a series of five probes were used, the first probe 0.3 m in front of the ice pack and the remaining four at distances of 0.3, 1.2, 2.4, and 3.7 m into the ice pack. The total length of the ice pack was 4.3 m. The first probe measured the combination of the incoming and reflected waves while the remaining four were used to measure the transmitted wave height. Because only one recording channel was available, a switching box was used to change the gage in the recording circuit at any time. At any particular wave setting, the open water wave was measured, followed by the other four gages. A minimum of 15-20 wave cycles were recorded at each gage for each wave setting to obtain an average wave height.

From the reduced data, the ratio of wave height to open water wave height (H/H_o) was determined as a function of x.

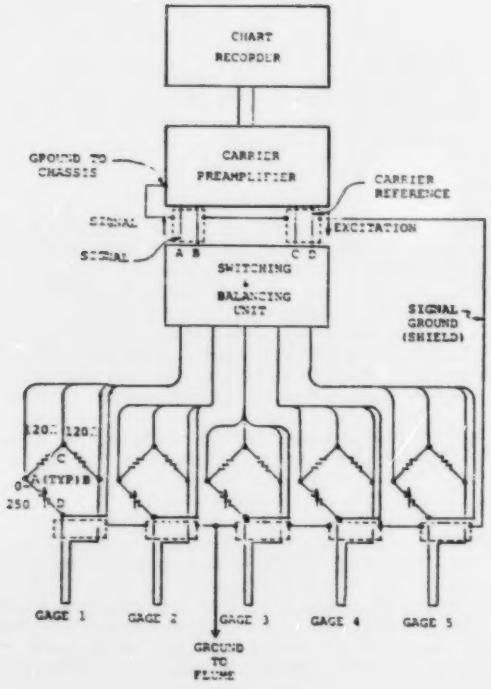


Figure 4. WAVE GAGE CIRCUIT DIAGRAM

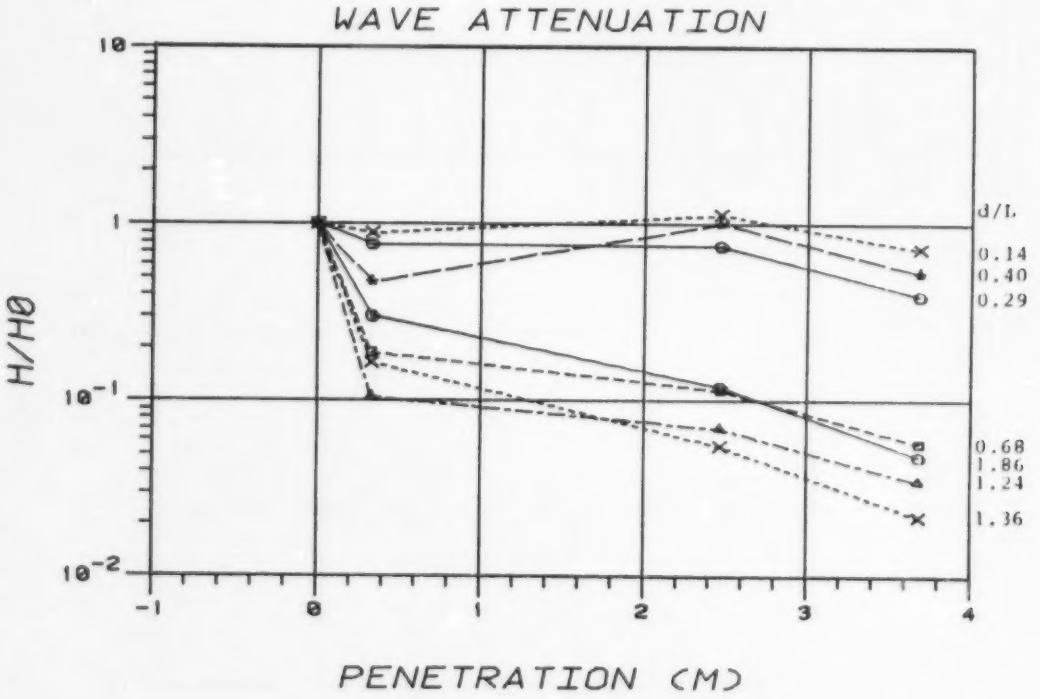


Figure 5. WAVE ATTENUATION WITH 9.52 CM STROKE

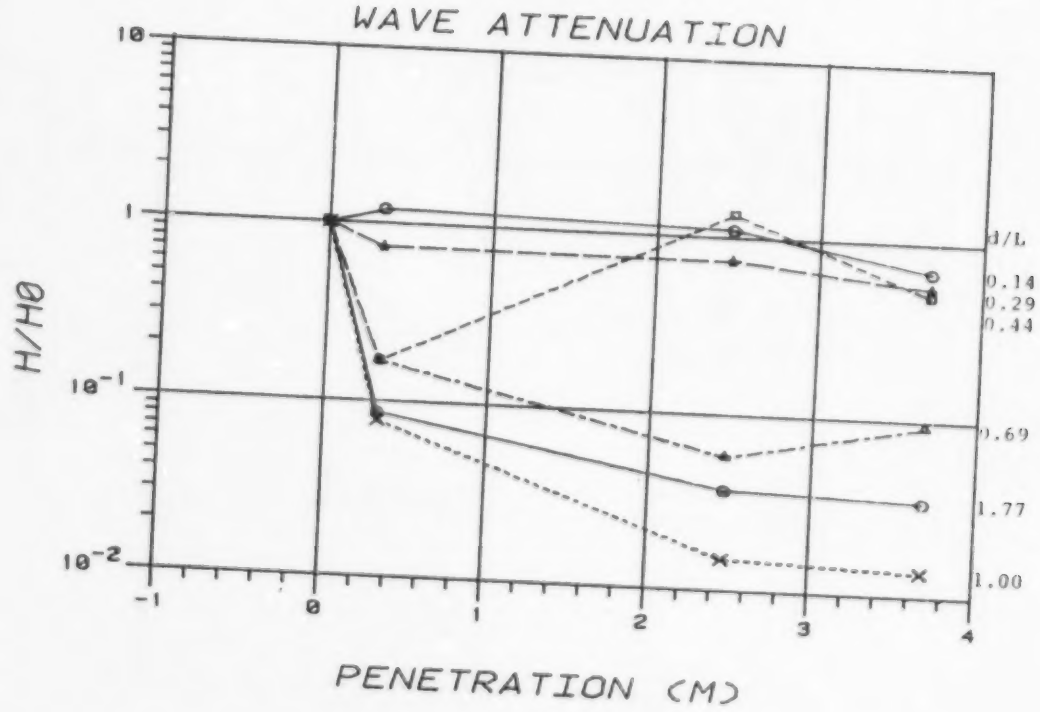


Figure 6. WAVE ATTENUATION WITH 19.68 CM STROKE

RESULTS

The wave attenuation rates in the ice packs are shown in Figs. 5 and 6.

As predicted by theory the attenuation rate is a function of d/L. It is also observed that the rate of attenuation decreases as d/L decreases. This is expected because the particle length, d, is related to the total mass of the particle for a given thickness and as the mass increases the decay terms become more prevalent. From the experimental data one can obtain a reasonable relationship in the form of

$$\ln(H/H_o) = -(\delta+\gamma x)$$

where $e^{-\delta}$ represents the transmission coefficient and γ is a decay or attenuation coefficient. This relationship was found to be

$$H = H_o \exp\{-[(0.0379+1.234(d/L)) + (0.1284+0.2134(d/L))x]\} \quad (16)$$

SUMMARY AND CONCLUSIONS

A combined theoretical and experimental investigation of surface ice-wave interaction produced a criteria for wave transmission and decay based on the physical properties of the ice and the approaching wave train. This relationship is given by Eq. 11.

The transmission coefficient is found to be a linear function of d/L:

$$\delta = 0.39 + 1.234\left(\frac{d}{L}\right) \quad (17)$$

as is the attenuation coefficient :

$$\gamma = 0.1284 + 1.2134\left(\frac{d}{L}\right) \quad (18)$$

Equations 16, 17, and 18 can be used to predict hydrodynamic wave loads on structures in the presence of a broken ice cover.

Further research into the combined effect of an elastic an a discrete-inelastic ice cover would be necessary to estimat wave loads in large ice fields like those encountered in th Arctic. The present solution provides satisfactory workin relationships for ice covers in rivers.

BIBLIOGRAPHY

1. Foltyn, E.P., "The Effect of a Broken Icefield on Free Surface Gravity Water Waves," submitted in partial fulfillment of the requirements for the degree of Master of Science, Dept. of Civil and Env. Eng., Clarkson University, Dec. 1982.

2. Havelock, T.H., Phil. Mag., Series 7, 8, 1929, 569-576.

3. Heins, A.E. and Feshbach, H., "The Coupling of Acoustic Ducts," Journal of Mathematics and Physics, No. 3, Vol. 26, Oct. 1947, 143-155.

4. Horikawa, K., "Coastal Engineering, An Introduction to Ocean Engineering," John Wiley & Sons, 1978.

5. Ippen, A., Editor, Estuary and Coastline Hydrodynamics, McGraw-Hill, 1966.

6. John, F., "On the Motion of Floating Bodies," Comm. on Pure and Applied Math., Vol. 3, No. 1, Mar. 1950, 45-101.

7. Keller, J.B. and Goldstein, E., "Water Wave Reflection Due to Surface Tension and Floating Ice," Transactions, American Geophysical Union, Vol. 34, No. 1, Feb. 1953, 43-48.

8. Keller, J.B. and Weitz, M. "Reflection and Transmission Coefficients for Waves Entering or Leaving an Icefield," Comm. on Pure and Applied Math., Vol. VI, 1953, 415-417.

9. Lamb, Sir H., Hydrodynamics, Sixth Edition, Dover Publications, 1945.

10. Newmann, J.N., Marine Hydrodynamics, MIT Press, 1980.

11. Peters, A.S., "The Effect of a Floating Mat on Water Waves," Comm. on Pure and Applied Math., Vol. 3, No. 4, Dec. 1950, 319-354.

12. Shapiro, A. and Simpson, L.S., "The Effect of a Broken Icefield on Water Waves," Transactions, American Geophysical Union, Vol. 34, No. 1, Feb. 1953, 36-42.

13. Stoker, J.J., Water Waves, The Mathematical Theory with Applications, Interscience Publishers, Inc., 1957.

14. Ursell, F., Dean, R.G. and Yu, Y.S., "Forced Small Amplitude Water Waves, A Comparison of Theory and Experiment," Journal of Fluid Mechanics 7, 33-52.

15. Wadhams, P., "Attenuation of Swell by Sea Ice," Journal of Geophysical Research, Vol. 78, No. 18, June 1973, 3552-3563.

16. Wadhams, P., "The Effect of a Sea Ice Cover on Ocean Waves," Ph.D. Dissertation, University of Cambridge, 1973, 223 pp.

17. Wadhams, P., "Airborne Laser Profiling of Swell in an Open Ice Field," Journal of Geophysical Research, Vol. 80, No. 33, Nov. 1975, 4520-4528.

18. Wadhams, P., "Wave Decay in the Marginal Ice Zone Measured from a Submarine," Deep Sea Research and Oceanographic Abstracts, Vol. 25, No. 1, Jan. 1978, 23-40.

19. Wadhams, P., "The Seasonal Ice Zone," NATO Advanced Study Institute on Air-Sea-Ice Interaction, Chapt. 10, Preliminary Report, Oct. 1981.

20. Weitz, M. and Keller, J.B., "Reflection of Water Waves from Floating Ice in Water of Finite Depth," Comm. on Pure and Applied Math., Vol. 3, 1950, 305-318.

DISCUSSION

Bruce Pratte

The experimental setup would have been better with simultaneous recording wave probes, and a scan along the channel to pick up reflection from the beach and from the ice cover.

(ie) to define a transmission coefficient the inter-particle friction is probably very important in determining propagation of waves through a broken ice cover. This experiment obviously had to compromise on ice block hinging and wave measurement. How would you prefer to carry out your tests if equipment was no problem? Would you not like to try growing a carbamide model ice cover at CRREL and scan its moving surface with say ultrasonic probes from above? There have been several papers in the literature on floating breakwaters, often made of tires roped-together in matress form. Perhaps some of the reflection and transmission coefficient results for them could be useful to you?

Reply

The writers would like to thank Dr. Pratte for his helpful questions and discussions on the experimental procedure which was conducted for the above referenced paper.

A better experimental setup would have been to use at least six wave probes, two of these being located in open water before the ice pack and the remaining probes spaced throughout the ice pack. More important than the number of probes however would be to read all probes simultaneously. Simultaneous recording on all probes would have allowed us to determine both reflection and transmission coefficients of an ice pack.

In addition, if one used an ultrasonic probe from above or a light and photo cell system from the side of the flume, one would have eliminated the physical interference between the wave probe and the flow field. In an open water situation, the gauge has little or no adverse effect on the wave characteristics, but with an ice cover one has to allow sufficient room for the probe so that it does not interfere with the cover.

Another change that could be made would have been a better choice of an aspect ratio. (i.e. floe thickness to floe length ratio). A representative floe size in the marginal ice zone in the Arctic Ocean would be a thickness of 3-5 m and a length of 50-100 m. This would give an aspect ratio of 0.03 to 0.1. The aspect ratio of our setup was 0.01. Furthermore the ratio of floe thickness to wave amplitude in the arctic is on the order of 1. Our setup had this ratio on the order of . These model scaling problems led to the model floes riding over one another until we fabricated a hinge to maintain their relative positions.

Finally performing these experiments in a cold room flume with a carbamide or urea doped ice cover. Using proper sizing ratios we would have been able to obtain more accurate results with fewer operational problems.

Further comparison with floating breakwater and sea ice attenuation results would be highly beneficial to the understanding of this phenomena.

PART TWO

ABSTRACTS FOR POSTER DISPLAYS

DEUXIEME PARTIE

SOMMAIRES DES AFFICHES A EXPOSER

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21,1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

NEW BRUNSWICK'S SUBCOMMITTEE ON RIVER ICE

Brian C. Burrell,[1] P. Eng.

ABSTRACT

Ice jams are a major cause of flooding in New Brunswick. In response to a need to coordinate and promote investigations of river ice in the Province, an ad hoc committee on river ice jams was established in 1978. In October of 1981, this ad hoc committee was replaced by the Subcommittee on River Ice, which was established under the Flood Forecasting Technical Committee as part of the Canada-New Brunswick Flood Damage Reduction Program.

Agencies represented on the Subcommittee on River Ice presently include the University of New Brunswick, the New Brunswick Electric Power Commission, the N.B. Departments of the Environment and Transportation, and Environment Canada.

The primary objectives of the Subcommittee on River Ice are (1) to develop methods to forecast the onslaught of ice breakups and jamming; (2) to document and analyze ice jamming in New Brunswick; and (3) to improve the reliability of winter discharge measurements and estimates by studying the effect of ice on streamflow. In order to accomplish these objectives, the member agencies on the Subcommittee on River Ice undertake or support field programs of measurements at identified sites on several rivers in the Province.

The secondary objectives of the Subcommittee are (1) to promote the coordination of ice measurement and data; (2) to promote the development of river ice engineering in New Brunswick; and (3) to promote the exchange of technical information and ideas. In regard to the first of the

---

[1]Hydrotechnical Engineer, Surface Water Section, Water Resources Branch, New Brunswick Department of the Environment

secondary objectives, the Subcommittee has adopted a standard form for ice measurements and established the Saint John River Forecast Centre as the depository of ice information for the Province. In regard to the other two secondary objectives, the Subcommittee assists in sponsorship of lecturers on river ice. It is also a sponsor of the Workshop on Hydraulics of River Ice being held in Fredericton on June 20-21, 1984.

The Subcommittee on River Ice has an operating budget of only a few thousand dollars, but has been effective due to the mutual cooperation and support of its member agencies. Certainly, much more could be accomplished if more funds were made available. However, the Subcommittee is an example of what can be accomplished if agencies work together on an area of common interest.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

SOUS-COMITÉ DU NOUVEAU-BRUNSWICK SUR LA GLACE DES COURS D'EAU

Brian C. Burrell [1], ingènieur

SOMMAIRE

Les embâcles de glace sont la cause principale des inondations au Nouveau-Brunswick. Pour satisfaire un besoin de coordination et de promotion des enquêtes menées sur la glace des cours d'eau de la province, un comité spécial sur les embâcles de glace des rivières a été créé en 1978. En octobre 1981, ce comité a été remplacé par le Sous-comité sur la glace des cours d'eau, qui a été mis sur pied sous la direction du Comité technique de prévision des inondations dans la cadre du Programme Canada-Nouveau- Brunswick de réduction des dommages dus aux inondations.

Les organismes représentés au sein du Sous-comité sur la glace des cours d'eau à l'heure actuelle comprennent l'Université du Nouveau-Brunswick, la Commission d'énergie électrique du Nouveau-Brunswick, les ministères de l'Environnement et des Transports de la province, ainsi qu'Environnement Canada.

Les objectifs principaux de ce sous-comité sont les suivants: l'élaboration de méthodes permettant la prévention du bris des glaces et de la formation d'embâcles, la documentation et l'analyse de la formation d'emblâcles au Nouveau-Brunswick, et l'amélioration de la fiabilité du mesurage de début hivernal et des prévisions grâce à l'étude des effets de la glace sur le courant des cours d'eau. En vue d'atteindre ces objectifs, les organismes membres du sous-comité entreprennent ou appuient des programmes

[1] Ingènieur en hydrotechnique, Section des eaux de surface, Direction de ressources en eau, ministère de l'Environnment du Nouveau-Brunswick

de mesurage sur le terrain à des endroits donnés sur plusieurs rivières de la province.

Les objectifs secondaires du sous-comité sont les suivants: la promotion de la coordination du mesurage des glaces et de la cueillette des données, la promotion du développement de l'ingénierie relative aux glaces recouvrant les cours d'eau du Nouveau-Brunswick, et la promotion de l'échanges d'informations et d'idées d'ordre technique. En ce qui a trait au premier des objectifs secondaires, le sous-comité a adopté une formule type pour le mesurage des glaces et fait du Centre de prévision des cures du fleuve Saint-Jean le dépôt de l'information de la province dans ce domaine. Pour ce qui est des deux autres objectifs secondaires, le sous-comité participe au parrainage de conférenciers sur la glace des cours d'eau. En autre, a commandité l'Atelier sur l'hydraulique des cours d'eau recouverts de glace, tenu à Frédericton, les 20 et 21 juin 1984.

Le sous-comité sur la glace des cours d'eau n'a qu'un budjet d'exploitation de quelques milliers de dollars, mais il a réussi à accomplir beaucoup grâce à la collaboration et l'appui mutuels de ses organismes membres. Il est certain qu'il pourrait faire beaucoup plus s'il disposait d'une plus grand quantité de fonds; toutefois, il est un exemple de ce que des organismes peuvent accomplir en collaborant ensemble dans un domaine d'intérêt commun.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

RIVER ICE INSTRUMENTATION DEVELOPMENTS AT THE NATIONAL WATER RESEARCH INSTITUTE

J. S. Ford [1], J. Valdmanis [1], A. S. Watson [1]

ABSRACT

Following the formation of the New Brunswick Sub-committee on River Ice in October 1981, the subcommittee suggested three specific instrument developments which could be undertaken by NWRI in the national interest. These were:

Ice thickness measuring device
Ice undersurface observation device
Frazil ice recorder

These instrument developments were initiated in 1982 by the Institute. The Under-ice Video System was completed for field evaluation in January 1983. System design of the frazil ice recorder system has progressed to the in-house test stage of a development model of the system. System development of an ice thickness measuring radar is in the research phase, with some propagation analysis and results obtained from the evaluation of a broad band ground probing radar applied to the ice environment.

The poster display describes the progress of these developments to-date, illustrating the configurations chosen, the actual or proposed methods for instrument installation and use, and the types of information it is intended these systems will be able to provide.

---

[1] Engineering Services Section, Hydraulics Division
National Water Research Institute
Canada Centre for Inland Waters, Burlington, Ontario, Canada L4R 4A6

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

MISES AU POINT D'INSTRUMENTS D'ANALYSE DE LA GLACE FLUVIALE A L'INSTITUT NATIONAL DE RECHERCHE SUR LES EAUX

J. S. Ford [1], J. Valdmanis [1], A. S. Watson [1]

SOMMAIRE

Après qu'il eut été formé en octobre 1981, le sous-comité du Nouveau-Brunswick sur la glace fluviale a suggéré la mise au point de trois instruments, qui pourrait être entreprise par l'Inre dans l'intérêt national:

Appareil servant à mesurer l'épaisseur de la glace
Appareil servant à observer la partie immergée de la glace
Appareil enregistreur du frasil.

Ces mises au point d'instruments ont été entreprises en 1982 par l'Institut. Le système vidéo sous-marin était prêt à être évalué sur le terrain en janvier 1983. L'étude technique de l'appareil enregistreur du frasil en est au stade des essais internes d'un modèle expérimental du système. La conception d'un radar mesurant l'épaisseur de la glace n'est à la phase des recherches, l'évaluation d'un radar à large faisceau servant à observer les terrains et s'appliquant à la glace ayant produit certains résultats et une analyse de la propagation des ondes.

L'affiche décrira l'état d'avancement de ces mises au point jusqu'à ce jour, illustrant les configurations choisies, les méthodes utilisées ou proposées d'installation et d'utilisation des instruments, les genres de renseignements que les systèmes sont censés être en mesure de fournir.

---

[1] Section des services de génie, Division de l'hydraulique, Institut national de recherche sur les eaux, Centre canadien des eaux intérieurs, Burlington, Ontario, Canada L5R 4A6

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

ICE JAMS IN NEW BRUNSWICK

Nabil Elhadi, P.Eng. [1] and George Richard [2]

ABSTRACT

Ice jams are one of the most striking of natural phenomenon in the Canadian Environment. They cause flooding and result in significant damages. The damages associated with ice jam events are inevitable consequences of man's encroachment upon the river floodplains for development and habitation.

In New Brunswick, ice jam events have been experienced for nearly 200 years, with the first ice jam ever recorded on April 25th, 1789. In this particular event, hillocks of ice were piled up 12 metres high at Maugerville downstream of the City of Fredericton. An examination of past records has indicated that the 1936 ice jam flood produced a water level at Fredericton which was approximately half a metre higher than that associated with the 1973 flood, the largest open water flood of record in the Province of New Brunswick.

The severity of ice jams events can be appreciated by glancing at the February 1970 flood, when unusual weather conditions in New Brunswick resulted in high water levels and heavy ice movements on most rivers. As a result, a total of 104 road bridges were damaged to some extent. Damage to 64 bridges were clearly a direct result of ice action; losses amounting to millions of dollars resulted due to damages to bridges alone.

This poster display depicts photographs of the 1936, 1970, 1976, and 1982 ice jam events in New Brunswick in order

[1] Chief Surface Water Section, Water Resources Branch, New Brunswick Department of the Environment
[2] Engineering Technician, Surface Water Section, Water Reources Branch, New Brunswick Department of the Environment

to demonstrate the devastating damages associated with ice jam flooding. It is, however, expected that the magnitude of ice jam flood damages will continually increase as the value of property subject to flooding in a river valley increases and as we continue to develop within the floodplains. Therefore, the efforts of planners and engineers need to be directed toward the proper management of development in flood prone areas and the undertaking of flood proofing measures.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

EMBACLES DE GLACE AU NOUVEAU-BRUNSWICK

Nabil Elhadi [1], ingénieur, et George Richard [2]

SOMMAIRE

Les emblâcles de glace sont le phénomène naturel le plus impressionnant de l'environnement canadien. Ils sont la cause d'inondations et entraînent des dommages considérables. Ces dommages sont les conséquences inévitables de l'empiètement de l'homme sur les plaines inondables des rivières aux fins d'aménagement de terrains et de construction d'habitations.

Des embâcles de glace se produisent au Nouveau-Brunswick depuis près de 200 ans et le premier ayant été enregistré est survenu le 25 avril 1789. Ce jour-là, les piles de glace atteignaient jusqu'à 12 mètres de hauteur à Maugerville, en aval de la ville de Fredericton. Selon un examen des vieux dossiers, l'inondations causée par un embâcle de glace en 1936 a entraîné un niveau d'eau à Fredericton d'environ un demi mètre de plus que celui associé à l'inondation de 1973, le plus grande inondation en eau libre jamais enregistrée dans la province du Nouveau- Brunswick. Il est possible d'évaluer la gravité des embâcles de glace en jetant un coup d'oeil à l'inondation de février 1970 lorsque des conditions météorologiques inhabituelles dans la province ont occasionné une hausse du niveau de l'eau et un accroissement considérable du mouvement des glaces sur la plupart des rivières. Ainsi, un total de 104 ponts routiers ont été plus ou moins endommagés. Le mouvement des glaces a été la cause directe des dommages causés à 64 ponts; ces dommages à eaux seuls ont entraîné des pertes de plusieurs millions de dollars.

---

[1] Chef de la Section des eaux de surface, Direction des ressources en eau, ministère de l'Environnement du Nouveau-Brunswick

[2] Technicien en sciences de l'ingénierie, section des eaux de surface, Direction des ressources en eau, ministère de l'Environnement du Nouveau-Brunswick

Cette affiche représente des photographies des embâcles survenus en 1936, 1970, 1976, et 1982 au Nouveau-Brunswick afin de démontrer les dommages dévastateurs associés aux inondations provoquées par ces embâcles. Cependant, on prévoit que l'étendue des dommages causés des inondations semblables continuera d'augmenter étant donné que la valeur des biens sujets aux inondations dans la vallée d'une rivière s'accroît et que nous continuous d'aménager des portions des plaines inonables. En conséquence, les planificateurs et les ingénieurs doivent orienter leurs efforts vers la bonne gestion de l'aménagement des zones sujettes aux inondations et la prise de mesures d'imperméabilisation.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21,1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

THE CANADIAN CLIMATE CENTRE ARCHIVE

J. Dublin [1], and S. F. Malone [1]

ABSTRACT

Ice grows, decays, breaks up and jams in rivers in response to weather. Research into these phenomena is aided by examining weather records for the rivers in question. The aim of this poster display is to give researchers in the field of ice research an opportunity to learn more about the Canadian Climate Archive. In particular, the data stored and their formats, methods of accessing the data, and programs available to obtain derived data or parameters are detailed.

[1] Scientific Services Unit, Atmospheric Environment Service, Environment Canada, Fredericton, NB.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

LES ARCHIVES DU CENTRE CLIMATIQUE CANADIEN

par J. Dublin [1] et S. F. Malone [1]

SOMMAIRE

La glace se forme, se détériore, se rompt et s'amoncelle dans les cours d'eau sous l'action des conditions météorologiques. Les recherches dans ce domaine sont facilitées par l'étude des relevés météorologiques pour les rivières et cours d'eau en cause. Ce montage a pour objet de donner l'occasion aux scientifiques qui font des recherches sur la glace de mieux connaître les Archives du Centre climatique canadien, en particulier les données consignées et leur forme, les méthodes de consultation des données ainsi que les programmes existant pour obtenir des données ou paramètres connexes.

[1] Services scientifiques, Service de l'environnement atmosphérique, Environnement Canada, Fredericton (N.-B.)

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

EXPLOITS RIVER FREEZE-UP AND BREAK-UP OBSERVATIONS 1984

by Douglas B. Hodgins [1] and H. Laban Davis [2]

ABSTRACT

Ice accumulations on the Exploits River in Newfoundland have contributed to 17 significant floods in the last 80 years. In January 1983, for example, ice and high discharges caused the Trans-Canada Highway to be flooded during the same event which destroyed part of the dam at Bishops Falls. A month later, flood damages approaching $100,000 arose at the community of Badger which was inundated as a result of ice jams during reformation of the ice cover.

A comprehensive program of field observations was subsequently initiated to evaluate the influence of ice on flooding along the Exploits River as part of a hydrotechnical study supported by the Canada-Newfoundland Flood Damage Reduction Program. Ice observations in 1984 were carried out from freeze-up to break-up and include 19 transect to define the time-varying change in ice thickness, frazil slush depth and river velocity beneath the cover. River discharge, backwater levels, water temperature, rate of cover progression and rate of decay were also monitored.

The results of this program are of significant value to future studies involving ice management and modelling. The observations are detailed and developed specifically for quantifying: the roughness of ice covers from formation

[1] Senior Water Resources Engineer, MacLaren Plansearch Inc., Toronto, Ontario
[2] Manager, Penco Newfoundland Ltd., St. John's, Newfoundland

to break-up; the thickness and location of frazil slush accumulations; hanging dams and cover forms in a variety of river locations; and the volume of frazil ice produced by open reaches of turbulent water.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU-BRUNSWICK

EXPLOITS COURS D'EAU DE LA GLACE ET LA DEBACLE

OBSERVATIONS 1984

Douglas B. Hodgins [1] et H. Laban Davis [2]

L'entassement des glaces sur Exploits River à Terre-Neuve a contribué dans les dernières 80 années à dix-sept inondations. En janvier 1983, par exemple, la crue des eaux et des glaces qui a détruit une partie du barrage de Bishops Falls, a également produit l'inondation sur la route trans-canadienne. Un mois plus tard, la localité de Badger a encouru des frais se chiffrant à près de $100,000 lorsque cette localité fut inondée par suite de l'accumulation de glaces pendant la période de reformation de la couche de glace.

Un programme d'observations, faisant partie d'une étude hydrotechnique appuyée par le Programme de la Réduction des Dommages Causés par les inondations Canada-Terre-Neuve, a été ultérieurement initié pour mesurer l'influence exercée par les glaces sur les inondations qui se sont produites le long de Exploits River. L'étude des glaces en 1984 fut conduite à partir du gel à 19 points stratégiques afin d'évaluer l'importance de la variation du temps dans la détermination de l'épaisseur de la glace, de l'épaisseur des glaces frazil, ainsi que de la vélocité des eaux sous le champs de glace. Le déversement, le niveau et la température de l'eau, le taux de la progression du champs de glace et le taux de sa désintégration ont été contrôlés.

Les résultats de ce programme sont d'une importance considérable sur les études subséquentes portant sur le maniement des glaces. Les observations sont détaillées et spécifiquement mises au point pour déterminer le facteur quantité dans: la rugosité, depuis la formation jusqu'au dégal; l'épaisseur et l'emplacement des glaces frazil, les barrages

[1] Ingérieur Supérieur Des Ressources En Eau
[2] Gérant, Fenco Newfoundland Ltd., St. John's Newfoundland

suspendus et la formation de couches de glace dans une variété de rivières déterminées; de même que la masse de champ de glace frazil produits par les étendues libres des eaux agitées.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU-BRUNSWICK

A FIELD SURVEY FORM FOR MONITORING ICE CONDITIONS ON LAKES AND RIVERS

T. D. Prowse [1] and J. C. Anderson [1]

ABSTRACT

Throughout most of Canada, ice is present on rivers and lakes for a large part of the winter season. Human activities can be greatly affected by freshwater ice such as through impacts on transportation, water supply, recreation and the like, but perhaps the most dramatic impact is the flooding which is caused by ice jamming. As the cost of compensating flood damage victims escalates, more attention is being focused upon ice conditions and in particular, river ice jamming and jam-induced flooding.

In order to gain a better understanding of ice conditions, research activity is increasing. Good field data are fundamental to this research effort and although some ice data bases already exist (e.g. Allen, 1977), they fall short of current requirements. A more comprehensive data base is needed if significant advances are to be made in our understanding of ice conditions.

The "Ice Survey Notes" form has been designed by the National Hydrology Research Institute in order to fulfill this recognized need for basic ice information. Though hopefully applicable to any river or lake, this form is intended primarily for use at sites where ice conditions are known to contribute significantly to high water levels and flooding. Data provided on the form will support studies of ice conditions such as the timing and character of freeze-up and/or break-up, and the occurrence and severity of ice jams. A long-term aim of such studies is the development of a forecasting capability for ice conditions.

[1] National Hydrology Section
National Hydrology Research Institute
Environment Canada, Ottawa

More specifically, the ice thickness and extent data provided on page one of the form will provide a data base for studies of ice growth, and the significance of ice thickness and ice types to jamming. The ice quality information on page two also contributes to studies of formation, decay and jamming. Information on pages three and four will support studies of the timing and character of freeze-up, break-up and jamming.

Allen, W.T.R., 1977. Freeze-up, break-up and ice thickness in Canada. Atmospheric Environment Service, Environment Canada, CLI-1-77.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOUVEAU BRUNSWICK

FORMULE D'OBSERVATIONS SUR L'ETAT DE LA GLACE DE LAC ET DE RIVIERE

T. D. Prowse [1] and J. C. Anderson [1]

SOMMAIRE

Pendant la plupart de l'hiver, la glace est présente sur presque tous les lacs et cours d'eau du Canada. La glace d'eau douce peut perturber grandement les activités de l'homme en raison de ses effets sur le transport, l'approvisionnement en eau, les loisirs, etc. Toutefois, l'effet le plus grave est peut-être celui des inondations causés par des embâcles. Comme les paiements versés en guise de dédommagement aux victimes d'inondations montent en flèche, on se préoccupe de plus en plus des conditions de la glace, notamment des embâcles et des inondations qui en résultent.

Par conséquent, les recherches se multiplient afin de mieux comprendre les conditions de la glace. Pour appuyer ces recherches, on a besoin de données fiables qui, bien qu'elles soient disponibles dans une certaine mesure (Allen, 1977, par exemple), ne suffisent pas à répondre aux besoins courants. Il s'avère donc essentiel d'établir une base de données plus complète si l'on veut faire progresser les connaissances relativement aux conditions de la glace.

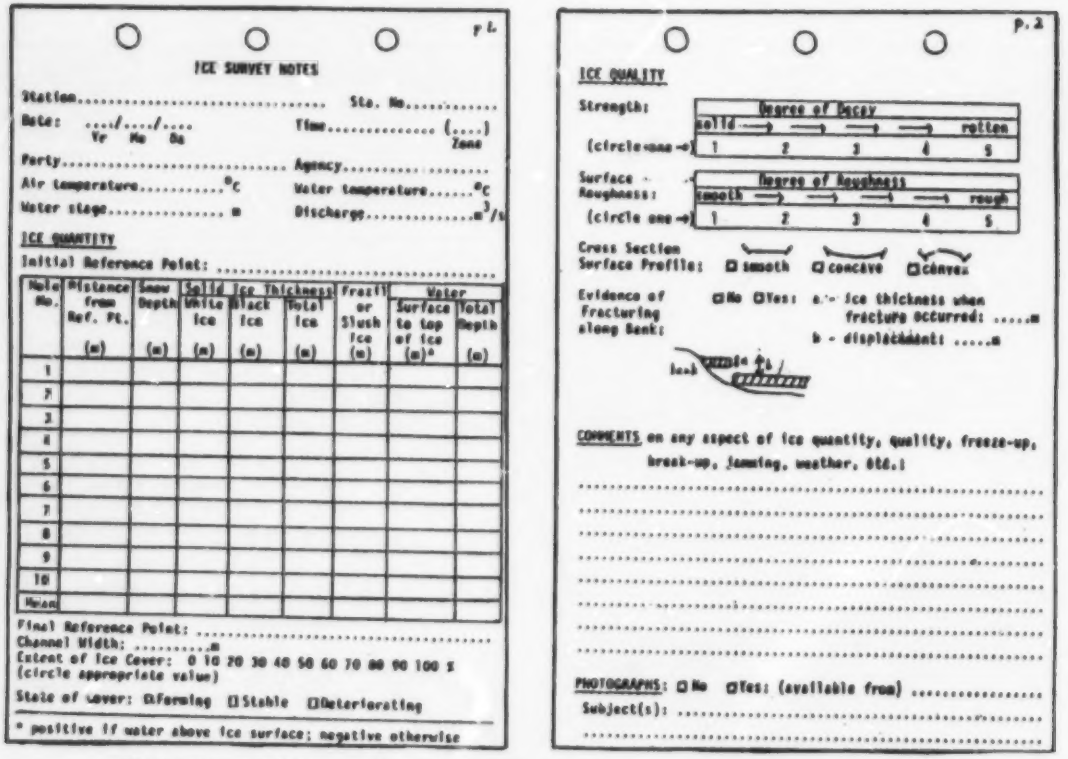
Pour ce faire, des employés de l'Institut national de recherches en hydrologie ont conçu la formule "Ice Survey Notes". Bien que son application devrait, espérons-le s'étendre à n'importe quel cours d'eau ou lac, elle devrait servir principalement aux endroits où la glace contribue grandement aux niveaux de crue et aux inondations. Les données fournies sur la formule faciliteront la tâche des chercheurs dans leurs études des conditions de la

[1] Section de l'hydrologie du Nord
Institut national de recherches en hydrologie
Environnement Canada, Ottawa

glace telles que la période et les caractérisques de la prise des glaces et de la débâcle ainsi que le fréquence et la gravité des embâcles. Un des buts à long terme de ces études est de mettre au point la capacité de prévoir les conditions de la glace.

Plus précisément, les données sur l'épaisseur et l'étendue de la glace qui sont notées à la page 1 de la formule, fourniront des renseignements nécessaires aux études sur le développement de la glace et l'importance de l'épaisseur et des types de glace par rapport à la formation d'embâcles. Les données portant sur la qualité de la glace à la page 2 contribueront également aux études de la formation et de la désintégration de la glace ainsi qu'a celles de la formation d'embâcles. Quant aux données inscrites aux pages 3 et 4, elles seront utiles pour les études relatives à la période à laquelle surviennent la prise des glaces, la débâcle et les embâcles de même qu'à leurs divers types.

Allen, W.T.R., 1977. Freeze-up, break-up and ice thickness in Canada. Atmospheric Environment Service, Environment Canada, CLI-1-77.



p.1

ICE SURVEY NOTES

Station..... Sta. No.....

Date: ..../..../.... Time..... (....)
Yr Mo Da Zone

Party..... Agency.....

Air temperature.....°C Water temperature.....°C

Water stage..... m Discharge.....m^3/s

ICE QUANTITY

Initial Reference Point: .....

Hole No.	Distance from Ref. Pt. (m)	Snow Depth (m)	Solid Ice: White Ice (m)	Ice Thickness: Black Ice (m)	Total Ice (m)	Frazil or Slush Ice (m)	Water: Surface to top of ice (m)*	Total Depth (m)
1								
2								
3								
4								
5								
6								
7								
8								
9								
10								
Mean								

Final Reference Point: .....
Channel Width: .....m
Extent of Ice Cover: 0 10 20 30 40 50 60 70 80 90 100 %
(circle appropriate value)

State of cover: ☐Forming ☐Stable ☐Deteriorating

* positive if water above ice surface; negative otherwise

p.2

ICE QUALITY

Strength: Degree of Decay

	solid →	→	→	→	rotten
(circle one →)	1	2	3	4	5

Surface Roughness: Degree of Roughness

	smooth →	→	→	→	rough
(circle one →)	1	2	3	4	5

Cross Section Surface Profile: ☐ smooth ☐ concave ☐ convex

Evidence of Fracturing along Bank: ☐No ☐Yes: a - ice thickness when fracture occurred: .....m
b - displacement: .....m

bank outside

COMMENTS on any aspect of ice quantity, quality, freeze-up, break-up, jamming, weather, etc.:

.....
.....
.....
.....
.....
.....
.....
.....

PHOTOGRAPHS: ☐No ☐Yes: (available from) .....
Subject(s): .....
.....

FIGURE 1. CURRENT DRAFT VERSION OF THE "ICE SURVEY NOTES", REDUCED FROM THE ACTUAL 13.5 x 20 cm PAGE SIZE.

p.3

FREEZE-UP

Event	Timing of Event ☐observed ☐estimated Yr	Mo	Da	Time	Water Stage (m)*
Initial ice formation					
Complete freeze-over					
Other / Comments:					

Presence of: ☐Border ice ☐Moving ice: (type(s)) .....

Icings? ☐No ☐Yes: covering ...% of surface cross section

On p.4, provide sketches of ice cover extent, type, etc.

BREAK-UP

Event	Timing of Event ☐observed ☐estimated Yr	Mo	Da	Time	Water Stage (m)*
Shore lead formation					
Transverse cracks or leads					
Main ice cover first moves					
Water clear of ice					
Other / Comments:					

ICE JAMS

Event	Timing of Event ☐observed ☐estimated Yr	Mo	Da	Time	Water Stage (m)*
Jam forms					
Jam exists					
Jam releases					

* ☐measured ☐estimated ☐above ☐below } which B.M.? .....

p.4

ICE JAMS (cont'd)

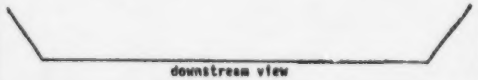
Jam type: ☐partial ☐floating ☐grounded: ☐at edge(s) ☐main channel

Jam length: .....km

In the space below, provide sketches of the channel cross section and reach to show: ice cover extent, height, type; heights of water below, at and above jam; extent of flooding

SKETCHES: For each sketch, include approximate scale.

1. River Cross Section



downstream view

2. River Reach (Use copy of map if available; show true north.)

FIGURE 1 (CONTINUED)

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

FREEZE-UP AND BREAK-UP PROCESSES ON THE YUKON RIVER IN THE VICINITY OF DAWSON CITY, YUKON TERRITORY

J. R. Janowicz [1]

ABSTRACT

Dawson City, Yukon Territory, is located on the floodplain of the Yukon River at the confluence of the Klondike River. During the Klondike Gold Rush of 1898, it was the largest city north of San Francisco and west of Winnipeg. It remains Yukon's main tourist center and the supply center for Yukon's present placer mining industry. Because of its historic significance the town is being restored by Parks Canada and private enterprise to capture and serve the growing tourist industry.

On May 3, 1979, an ice jam related flood caused extensive damage to the community which resulted in the dispensation of approximately two million dollars in flood benefits by the federal government. Subsequent to that event two years of freeze-up and break-up observations were carried out, and all available historical information was reviewed under the auspices of the Yukon River Basin Study. A major objective of the study was to assess the influence of hydroelectric development on the downstream ice regime of the Yukon River. Of specific concern was the effect of such a development on the break-up process in the Dawson area.

The available historical information has been summarized and the relationship between the freeze-up and break-up processes has been examined. These recent activities help provide a better understanding of the ice regime of the Yukon River and will hopefully lead to the alleviation of flooding at Dawson.

The display will focus on the 1979 break-up, a major flood year, and subsequent observations and studies.

[1] Hydrologist, Indian and Northern Affairs Canada
Northern Affairs Program, Water Resources Division

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

PRISE DE LA GLACE ET LA DEBLACE SUR LA RIVIÈRE DU YUKON DANS LES ENVIRONS DE LA VILLE DE DAWSON, TERRITOIRE DU YUKON

J. R. Janowicz [1]

SOMMAIRE

Dawson, au Yukon, est située dans la plaine inondable du fleuve Yukon, au confluent de la rivière Klondyke. Au temps de la ruée vers l'or du Klondyke en 1898, Dawson était la plus grande ville au nord de San Francisco et à l'ouest de Winnipeg. Aujourd'hui, Dawson demeure le principal centre touristique du Territoire ainsi que le centre d'approvisionnement de l'industrie yukonaise de l'exploitation des métaux alluvionnaires. Parcs Canada et l'entreprise privée travaillent à la restauration de la ville, en raison de son importance historique, afin d'attirer les touristes et de contribuer à l'essor de l'industrie touristique.

Le 3 mai 1979, une crue d'embâcle a causé d'importants dégâts dans cette localité et le gouvernement fédéral a dû verser quelque deux millions de dollars en presentation pour venir en aide aux sinistrés. Dès lors, on a observé pendant deux années consécutives la prise de la glace et la débâcle; tous les renseignements antérieurs dont on disposait sur le sujet ont également été étudiés dans le cadre de l'Etude du bassin de la rivière Yukon, dont un des principaux objectifs était d'évaluer l'incidence des aménagements hydroélectriques sur le régime des glaces du cours inférieur du fleuve Yukon. Dans cet exercice, l'un des points les plus importants était la détermination de l'effet d'un aménagement hydroélectrique sur la processus de débâcle dans la région de Dawson.

---

[1] Hydrogéologue, Ministère des affairs indiennes et du nord. Programme des affaires du nord. Division des ressources en eau.

Les données historiques que l'on possédait ont été résumées et les rapports entre les processus de prise de la glace et de débâcle ont été examinés. Ces travaux contribuent à faire mieux comprendre le régime des glaces du fleuve Yukon et, on l'espère, à réduire des inondations à Dawson.

Le montage porte sur la débâcle de 1979, année où l'inondation a été très forte, et les observations et étude qui ont été faites depuis.

WORKSHOP ON THE HYDRAULICS OF RIVER ICE
JUNE 20 AND 21, 1984
FREDERICTON, NEW BRUNSWICK

ATELIER SUR L'HYDRAULIQUE DE LA GLACE DES COURS D'EAU
LES 20 ET 21 JUIN 1984
FREDERICTON, NOVEAU BRUNSWICK

THE NRCC WORKING GROUP ON RIVER ICE JAMS

S. Beltaos [1]

ABSTRACT

The Working Group on River Ice Jams is affiliated with the NRCC Subcommittee on Hydraulics of Ice Covered Rivers and the Associate Committee on Hydrology. The Group members are: S. Beltaos (Chairman), National Water Research Institute; D. Andres, Alberta Research Council; B. Burrell, Environment New Brunswick; R. Gerard, University of Alberta; R. Halliday, Water Survey of Canada; F. Parkinson, La Salle Hydraulics Laboratory Ltd,; S. Petryk, Rousseau Sauvé Warren Inc.; and T. Prowse, National Hydrology Research Institute.

Ice jamming is a major hydrotechnical problem in Canada and has serious repercussions to flooding, design of river structures, erosion, interference with navigation, etc. The available technology to deal with such problems is limited and relies heavily on historical information and experience. This is partly due to the complexity of ice jamming phenomena but the situation is compounded by lack of systematic field observations and quantitative data. The most pressing immediate needs are thus perceived to include development of guidelines for field data collection programs and for utilization of existing data; assessment of research needs; and systematic compilation of past experience. In the longer term, a Monograph on Ice Jams would be a timely contribution.

To address the perceived needs, the Working Group has defined the following tasks.

1. To develop guidelines for needed data collection programs. There is increasing awareness that

[1] Research Scientist, Hydraulics Division
National Water Research Institute, Burlington, Ontario

improvement of the state of the art largely depends on accumulation of field data but progress is hampered by a general lack of concrete guidelines on what data should be collected and why. Guidelines are now under preparation for the use of agencies that may be interested in implementing ice monitoring programs.

2. To develop guidelines for utilizing data that are already available. Implementation of data collection programs, such as envisaged in Task I, will help develop an adequate data base but realization of the benefits will require several years after initiation. In the meantime, existing sources can be considered, such as hydrometric gauge data obtained by Water Survey of Canada and other agencies throughout a national gauge network. This source is imperfect but accessible at little cost and adequately covers the Canadian ranges of stream and weather types. Preparation of Guidelines for extracting ice-related information from gauge records is nearing completion.

3. To identify research needs. Work on this task is almost complete and a paper is to be presented at the Third Workshop on Hydraulics of River Ice.

4. To promote documentation and compilation of case studies. On many rivers experiencing ice jam problems, engineering studies have been carried out by entities concerned with planning, design, construction and operation of river development projects. A compilation of case studies is now progress and will be published in loose-leaf format to facilitate updating. An introductory paper is to be presented at the Third Workshop on Hydraulics of River Ice.

5. To prepare a monograph on ice jams. The Working Group has identified a need for a monograph comprising a comprehensive summary of current knowledge and broad design guidelines. After completion of a Table of Contents, the Group will seek the parent subcommittee's approval to perform this task.

CLOSING REMARKS

To all of you who have come from different parts of Canada and the U.S.A., a warm thanks in behalf of the General Organizing Committee. Your enthusiastic support and active participation have in the main contributed to the success of this Workshop.

The work of the General Organizing Committee was always a smooth and pleasant task largely due to the magnificent work of the following Subcommittees:

Technical Program Subcommittee
(Chairman, B. Burrell)

Registration and Budget Subcommittee
(Chairman, D.I. Bray)

Social Events Subcommittee
(Chairman, J.G. Lockhart)

Membership in each of these subcommittees is given elsewhere. It is appropriate to acknowledge the sustained and most effective work of members of the General Organizing Committee; its spirit of teamwork, capacity for swift problem solving and quiet efficiency were indeed marvellous. Their efforts were a constant source of inspiration to the Chairman and it is time to offer warm thanks for their extended effort.

Participants are requested to give their assessment of the 3rd Workshop on the Evaluation Forms; these assessments will be of considerable help for reviewing the goals and achievements of this Workshop and preparing for the next one.

The 4th Workshop and Short Course are to be held in Montreal in June 1986; Dr. Marc Drouin is the Chairman and Dr. Sylvester Petryk, Co-Chairman of the Organizing Committee. May I say Bonne Chance Montreal.

Finally, may I wish all of you a safe trip home, with memories of an enjoyable and productive 3rd Workshop on the Hydraulics of River Ice.

K.S. Davar, Chairman
General Organizing Committee

ALLOCUTION DE CLOTURE

Au nom du Comité organisateur, je tiens à remercier sincèrement tous les participants, qui sont venus d'un peu partout au Canada et aux Etats-Unis. Leur soutien et leur participation ont grandement contribué à faire de cet Atelier une réussite.

Le Comité organisateur a eu la tâche facile grâce a l'excellente collaboration:

du Sous-comité du programme technique, présidé par B. Burrell;

du Sous-comité de l'inscription et du budget, présidé par D.I. Bray;

du Sous-comité du programme social, présidé par J.G. Lockhart.

Le nom des membres de chacun de ces sous-comités est donné ailleurs. L'occasion se prête bien pour signaler le travail incessant et des plus efficaces des membres du Comité organisateur et pour rendre hommage à leur espirit d'équipe, à leur capacité de résoudre rapidement les problèmes ainsi qu'à leur grande efficacité. Le Président a tiré de leurs efforts une source intarissable d'inspiration et le moment est venu de remercier sincèrement ses membres de leurs efforts soutenus.

Les participants voudront bien donner leur appréciation du 3e Atelier en remplissant les Fiches d'évaluation. Ces Fiches seront très utiles pour effectuer l'examen des objectifs et des réalisations de cet Atelier ainsi que pour preparer le prochain Atelier.

Le 4e Atelier et le Programme abrégé se tiendront à Montréal en Juin 1986. MM. Marc Drouin et Sylvester Petryk sont respectivement président et coprésident du Comité organisateur. Permettez-moi de souhaiter bonne chance à Montréal.

En terminant, je vous souhaite à tous un bon retour et j'espère que vous garderez un bon souvenir du 3e Atelier sur l'hydraulique de la glace des cours d'eau.

Le Président du Comité organisateur,
K.S. Davar

ANNEXE A - ORGANISATEURS DE L'ATELIER

APPENDIX A - WORKSHOP SPONSORS

ANNEXE A

ORGANISATEURS DE L'ATELIER

Sous-comité du C.N.R. sur l'hydraulique des cours d'eau recouverts de glace

Le Sous-comité du Conseil national de recherches sur l'hydraulique des cours d'eau recouverts de glace est conjointement affilié au Comité associé sur l'hydrologie et au Comité associé sur la recherche en géotechnique. À l'origine, le Sous-comité a été formé en 1975 sous le nom du Groupe de travail du C.N.R. sur l'hydraulique des cours d'eau recouverts de glace. Comme ses programmes et ses activités ont évolué un une sphère d'activités courantes qui se poursuivront probablement encore longtemps, le statut de ce groupe de travail a été changé à celui de sous-comité en 1981.

Le Sous-comité sur l'hydraulique des cours d'eau recouverts de glace s'intéresse principalement à l'identification, à l'évaluation et à la coordination des travaux de recherches et d'élaboration dans le domaine de la glace recouvrant les cours d'eau.

L'un des principaux champs d'action du Sous-comité est l'organisation des ateliers portant sur la glace recouvrant les cours d'eau. Il s'agit du troisième atelier. Le premier a été organisé à Burlington au mois de septembre 1980 et le deuxième à Edmonton en juin 1982.

Sous-comité du N.-B. sur la glace des cours d'eau

Le Sous-comité du N.-B. sur la glace des cours d'eau est un sous-comité officieux formé dans le cadre du Programme Canada-Nouveau-Brunswick de réduction des dommages dus aux innondations. Ses objectifs sont les suivants:

- élaboration de méthodes permettant la prévention du bris des glaces et de la formation d'embâcles;
- documentation et analyses portant sur la formation d'embâcles au Nouveau-Brunswick;
- étude des effets de la glace sur le courant des cours d'eau;
- promotion du développement de l'ingénierie relative aux glaces recouvrant les cours d'eau au Nouveau-Brunswick;
- coordination de mesurage des glaces et des efforts en matière de cueillette des données au Nouveau-Brunswick; et
- promotion de l'échange d'information et d'idées d'ordre technique entre les différents organismes.

Les organismes suivants sont représentés au sein du Sous-comité:

Institut national de recherche sur les eaux, Environnement Canada
Institut national de recherche en hydrologie, Environnement Canada
Direction des ressources hydrologiques, Environnement Canada
Service de l'environnement atmosphérique, Environnement Canada
Ministère de l'Environnement du Nouveau-Brunswick
Commission d'énergie électrique du Nouveau-Brunswick
Ministère des Transports du Nouveau-Brunswick
Université du Nouveau-Brunswick

APPENDIX A

WORKSHOP SPONSORS

N.R.C. Subcommittee on Hydraulics of River Ice

The National Research Council's Subcommittee on Hydraulics of River Ice is jointly affiliated with the Associate Committee on Hydrology and the Associate Committee on Geotechnical Research. The Subcommittee was originally formed in 1975 as the N.R.C. Working Group on Hydraulics of River Ice. Since its programs and activities evolved to ongoing activities which are likely to continue for a considerable time into the future, the status of the working group was changed to that of a subcommittee in 1981.

The Subcommittee on Hydraulics of River Ice is primarily concerned with the identification, assessment and coordination of research and development work in the field of river ice.

One of the Subcommittee's main activities is the sponsorship of workshops on river ice. This is the third in the series of workshops. The first workshop was held in Burlington during September 1980 and the second workshop was held in Edmonton in June 1982.

N.B. Subcommittee on River Ice

The New Brunswick Subcommittee on River Ice is an informal subcommittee established as part of the Canada-New Brunswick Flood Damage Reduction Program. Its objectives are:

- to develop methods to forecast the initiation of ice breakup and jamming;
- to document and analyze ice jamming in New Brunswick;
- to study the effects of ice on streamflow;
- to promote the development of river ice engineering in New Brunswick;
- to coordinate ice measurement and data collection efforts in New Brunswick; and
- to promote the exchange of technical information and ideas between various agencies.

Represented on the Subcommittee are the following agencies:

National Water Research Institute, Environment Canada
National Hydrology Research Institute, Environment Canada
Water Resources Branch, Environment Canada
Atmospheric Environment Services, Environment Canada
New Brunswick Department of the Environment
New Brunswick Electric Power Commission
New Brunswick Department of Transportation
University of New Brunswick

Université du Nouveau-Brunswick

Le premier cours en ingénierie, offert par une université canadienne, a été donné le 15 février 1854 à l'Université du Nouveau-Brunswick qui à cette époque, s'appelait King's College. Vingt-six étudiants mâles y étaient inscrits.

Aujourd'hui, l'Université du Nouveau-Brunswick est une institution mixte et moderne offrant une variété de programmes en Arts et en Sciences à plus de 5 000 étudiants. Sir Edmund Head Hall, où sont offerts les ateliers, comporte des salles de classe et des laboratoires utilisés dans le cadre de sept programmes d'ingénierie accrédités: le génie chimique, civil, électrique, forestier, géotechnique, mécanique et d'arpentage ainsi qu'une grande école de science informatique. Aujourd'hui, l'Université du N.-B. compte dans ses rangs plus de 150 étudiants inscrits en ingénierie.

University of New Brunswick

On February 15, 1854, the first course of study in Engineering at a Canadian University was begun at the University of New Brunswick (then King's College). Twenty-six male students were enrolled.

Today, the University of New Brunswick is a modern co-educational institution offering a variety of programs in the arts and sciences to over 5,000 students. Sir Edmond Head Hall, where the workshop was held, contains classroom and laboratories for seven accredited engineering programs: Chemical, Civil, Electrical, Forest, Geotechnical, Mechanical and Surveying Engineering - and a large School of Computer Science. Today, there are over 150 engineering students.

ANNEXE B - LISTE DES PARTICIPANTS

APPENDIX B - LIST OF PARTICIPANTS

LIST OF REGISTRANTS

3rd WORKSHOP ON THE HYDRAULICS OF RIVER ICE
(June 20-21, 1984)

University of New Brunswick

Razek Abdelnovr
Vice President
Arctec Canada Ltd.
311 Legget Dr.
Kanata, Ontario
K2K 1Z8 (613)592-2830

David Andres
Research Officer
Alberta Research Council
4th Floor
4445 Clagary Trail South
Edmonton, Alberta
T6H 5R7 (403)438-1555

Spyros Beltaos
Research Scientist
National Water Research Institute
P.O. Box 5050
Burlington, Ontario
L7R 4A6 (416)637-4329

Denise A. Bunte-Bisnett
Civil-Hydraulic Engineer
New York Power Authority
P.O. Box 700
Massena, New York
13662 (315)764-0226

Darryl Calkins
Research Hydrotechnical Engineer
USA CRREL
72 Lyme Rd.
Hanover, New Hampshire
03755 (603)646-4325

Philip Cheung
Hydrotechnical Engineer
Proctor & Redfern Ltd.
45 Green Belt Drive
Don Mills, Ontario
M3C 3K3 (416)445-3600

John C. Anderson
Head, Cold Regions Section
National Hydrology Res. Inst.
Surface Water Division
Environment Canada
Ottawa, Ontario
K1A 0E7 (809)997-2385

R. Beauchemin
Evaluation Officer
Environment Canada
Room 923,
351 St. Joseph Blvd.
Hull, Quebec
K1A 0E7 (613)997-2639

Dale I. Bray
Professor
Dept. of Civil Engineering
University of New Brunswick
P.O. Box 4400
Fredericton, New Brunswick
E3B 5A3 (506)453-4521

Brian Burrell
Hydrotechnical Engineer
Water Resources Branch
P.O. Box 6000
Fredericton, New Brunswick
E3B 5H1 (504)453-2353

S.P. Chee
Professor and Head
Dept. of Civil Engineering
University of Windsor
Windsor, Ontario
N9B 3P4 (519)253-4232

Kersi S. Davar
Professor
Dept. of Civil Engineering
University of New Brunswick
P.O. Box 4400
Fredericton, New Brunswick
E3B 5A3 (506)453-4521

Jean-Guy Deveau
Area Engineer
Water Survey of Canada
Inland Waters Directorate
633 Queen Street
Fredericton, New Brunswick
E3B 1C3 (506)452-3095

Marc Drouin
Head, Hydraulic Department
James Bay Energy Corp.
800 Est. Boul. Maisonneuve
Montréal, Québec
H2L 4M8 (514)844-3741

Stephane Dumont
Great Lakes - St. Lawrence
Study Office
Environment Canada
200 - 45 Second St. East
Cornwall, Ontario
K6H 5R8 (613)932-4325

Edward P. Foltyn
Water Systems Engineer
Onondaga County Water Authority
16 Glenlynn Ct.
Liverpool, New York
13088 (315)673-4412

Richard Fontaine
Hydrologist
U.S. Geological Survey
26 Ganneston Dr.
Augusta, Maine
04330 (207)622-8208

Derek M. Foulds
R.R. # 4
Uxbridge,
Ontario
(416)852-6416

Robert Gerard
Professor
Dept. of Civil Engineering
University of Alberta
Edmonton, Alberta
T6G 2G7 (403)432-2066

P.J. Doyle
Senior Engineer
Trans Alberta Utilities
P.O. Box 1900
Calgary, Alberta
T2P 2M1 (403)267-7382

John Dublin
Meteorologist
Scientific Services Unit
Atmospheric Environment Serv.
633 Queen St.
Fredericton, New Brunswick
E3B 5W9 (506)452-3837

Nabil Elhadi
Chief, Surface Water Section
Water Resources Branch
N.B. Dept. of Environment
P.O. Box 6000
Fredericton, New Brunswick
E3B 5H1 (506)453-2353

Fernando Fonseca
Hydro-Quebec
Place Dupuis
855 rue Ste-Catherine est
Montreal, Québec
H2L 4N4 (514)289-5527

J.S. Ford
Project Engineer
National Water Research Inst.
P.O. Box 5050
Burlington, Ontario
L7R 4A6 (416)637-4293

Paul Frigon
Water Resources Engineer
Grand River Conservation Auth.
400 Clyde Road, Box 729
Cambridge, Ontario
N1R 5W6 (519)621-2761

Ian Gillis
Civil Engineering Student
University of New Brunswick
4 Elmcroft Place
Fredericton, New Brunswick
E3B 1Y8 (506)455-8051

D.G. Hayward
Manager
System Planning Division
N.B. Electric Power Commission
515 King Street
Fredericton, New Brunswick
E3B 4X1 (506)453-4246

Jimmy Ho
Graduate Student
Dept. of Civil Engineering
University of New Brunswick
P.O. Box 4400
Fredericton, New Brunswick
E3B 5A3 (506)453-4521

Sayed Ismail
Water Resources Engineer
N.B. Electric Power Commission
515 King Street
P.O. Box 2000
Fredericton, New Brunswick
E3B 4X1 (506)453-4283

J.N. Jasper
Senior Hydrologist
Water Resources Division
Indian & Northern Affairs
P.O. Box 1500
Yellowknife, NWT
K1A 2R3 (403)920-8249

Bruce W. Kitchen
Water Control Engineer
Trent Severn Waterway
P.O. Box 567
Peterborough, Ontario
K9J 6Z6 (705)742-9267

Roy Lane
Hydrometric Supervisor
Water Survey of Canada
Inland Waters Directorate
633 Queen St.
Fredericton, New Brunswick
E3B 1C3 (506)452-3095

Alan S. Lawrence
Office Engineer
N.B. Electric Power Commission
515 King Street
Fredericton, New Brunswick
E3B 4X1 (506)453-3041

Frank Hendriksen
Graduate Student
Dept. of Civil Engineering
University of New Brunswick
P.O. Box 4400
Fredericton, New Brunswick
E3B 5A3 (506)453-4521

Doug Hodgins
Sr. Water Resources Engineer
Maclaren Plansearch
33 Yonge St.
Toronto, Ontario
M5E 1E7 (416)365-7275

Ric Janowicz
Hydrologist
Northern Affairs Program
Indian & Northern Affairs
200 Range Rd
Whitehorse, Yukon
Y1A 3V1 (403)668-5151

Terry Kemp
Research Engineer
ESSO Resources Canada
330-50 Ave. SE
Calgary, Alberta
T2G 2B3 (403)259-0305

Jacques Labbé
Chef Service des Inventaires
Ministère de l'Environnement
-Quebec
917, Mgr. Grandin
Ste-Foy, Québec
G1V 3X8 (418)643-3461

S. Thomas Lavender
Senior Hydraulic Engineer
Acres Consulting Services
5259 Dorchester Road
P.O. Box 1001
Niagara Falls, Ontario
L2E 6W1 (416)354-3831

J.G. Lockhart
Director, Saint John River
Forecast Centre
N.B. Dept. of Environment
P.O. Box 6000
Fredericton, New Brunswick
E3B 5H1 (506)453-2353

Ian MacGougan
Research Assistant
Dept. of Civil Engineering
University of New Brunswick
P.O. Box 4400
Fredericton, New Brunswick
E3B 5A3 (506)453-4521

Maurice Miron
Chef Div. Hydrometrie
Ministère de l'Environnement
-Québec
2345 Dalton
Ste-Foy, Québec
G1P 3S3 (418)643-4553

U.S. Panu
Head, Hydrologic Modeling Branch
Water Resources Division
Nfld. Dept. of Environment
Elizabeth Towers, P.O. 4570
St. John's, Newfoundland
A1C 5T7 (709)737-2535

Richard Penn
River Control Engineer
Ontario Hydro
700 University Ave.
Toronto, Ontario
M5X 1A3 (416)592-2895

Bruce Pratte
Research Officer
Hydraulics Laboratory
National Research Council
Montreal Road
Ottawa, Ontario
K1A 0R6 (613)993-9201

Dieter Rickermann
Graduate Student
Dept. of Civil Engineering
University of New Brunswick
P.O. Box 4400
Fredericton, New Brunswick
E3B 5A3 (506)453-4521

Bernard Michel
Professor
Dept. Genie Civil
Universite Laval
Ste-Foy, Quebec
G1K 7P4 (418)656-2205

Jean Monfet
Dir. desouvrages hydrauliques
Ministère de l'Environnement
-Québec
3900 rue Marly
Ste-Foy, Québec
G1X 4E4 (418)644-6604

Fred E. Parkinson
Vice President
Lasalle Hydraulic Laboratory
0250 St. Patrick
Lasale, Québec
H8R 1R8 (514)366-2970

Syl Petryk
Senior Engineer
Rousseau, Sauve, Warren Inc.
1155 Metcalf St.
Suite 840
Montréal, Québec
H3B 2V6 (514)878-2621

Bruce A. Reid
Water Management Coordinator
Rideau Valley Conservation Auth
P.O. Box 599
Manotick, Ontario
K0A 2N0 (613)692-3571

Gilles Rivard
Graduate Student
Dept. of Civil Engineering
University of Alberta
Edmonton, Alberta
T6G 2M7 (403)432-3383

Sylvain Robert
Ingenieur Service Production
Hydro Quebec
75 Dorchester Street
Montréal, Québec
H2Z 1A4 (514) 289-4098

Hung Tao Shen
Professor
Dept. of Civil and
Environmental Engineering
Clarkson University
Potsdam, New York
13676 (315) 268-6606

David Smith
Inland Waters Directorate
Environment Canada
5th Floor, Queen Sq.
45 Alderney Dr.
Dartmouth, Nova Scotia
B2Y 2N6 (902) 426-3266

Patrick Tang
Flood Forecast Engineer
N.B. Dept. of Environment
P.O. Box 6000
Fredericton, New Brusnwick
E3B 5H1

Arun Valsangkar
Associate Professor
Dept. of Civil Engineering
University of New Brunswick
P.O. Box 4400
Fredericton, New Brunswick
E3A 5A3 (506) 453-4521

Glendon M. Wilson
Production Manager
N.B. Electric Power Comm.
P.O. Box 1260
Grand Falls, New Brunswick
G0J 1M0 (506) 473-1729

James Wuebben
Research Hydraulic Engineer
USA CRREL
72 Lyme Road
Hanover, New Hampshire
03741 (603) 646-4100

Henry S. Santeford
Associate Professor
Dept. of Civil Engineering
Michigan Technological Univ.
Houghton, Michigan
49931 (906) 487-2529

Bernard E. Shumilak
Hydraulic Engineer
Hydro Development Department
Manitoba Hydro
P.O. Box 815
Winnipeg, Manitoba
R3C 2P4 (204) 474-4191

David Sullivan
Bridge Hydraulics Engineer
N.B. Dept. of Transportation
P.O. Box 6000
Fredericton, New Brunswick
E3B 5H1 (506) 453-2674

Pierre-Richard Tremblay
Chef Service hydraulique/
hydrologie
ABBDL-TECSULT
85 rue Ste-Catherine ouest
Montréal, Québec
H2X 3P4 (514) 287-8557

A.R. Waroway
Engineering and Planning
Water Resources Division
Indian & Northern Affairs
Ottawa, Ontario
K1A 0H4 (613) 997-0559

Jonathan Wong
River Ice Research Engineer
National Water Research Inst.
867 Lakeshore Road
P.O. Box 5050
Burlington, Ontario
L7R 4A6 (416) 637-4274

Jon Zufelt
Rsearch Civil Engineer
USA CRREL
72 Lyme Road
Hanover, New Hampshire
03741 (603) 646-4225

Exploits River
ce Observations